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1. McNichol, Dan, *Paving the Way: Asphalt in America,* National Asphalt Pavement Association, Report HB-3, 2005.
2. Highway Statistics 2005, Federal Highway Administration, U. S. Department of Transportation, 2006.

CHAPTER 2. ASPHALT REFINING, USES, AND PROPERTIES

HISTORICAL BACKGROUND AND REFINING

Historical Background

Asphalt cement and tar are considered to be bituminous materials. Quite often, these two terms are used interchangeably due to misconceptions resulting from their similarity in appearance and in some parallel applications. However, asphalt cement and tar are two distinctly different materials with different origins and different chemical and physical characteristics. Asphalt cement is a dark brown to black cementitious material that is either naturally occurring or is produced by petroleum distillation. Tar, on the other hand, is primarily manufactured from the destructive distillation of bituminous coal and has a very distinct odor. Asphalt cement is used principally in the United States in paving applications. Tar is seldom used in paving in the United States except as a sealer because of (a) some undesirable physical characteristics such as very high temperature susceptibility, and (b) health hazards such as severe eye and skin irritation when exposed to its fumes. As a result, the focus of this book is on asphalt mixtures but much of what is said about the properties of asphalt mixtures will also apply to tar mixtures.

Asphalt cement is one of man's oldest engineering materials. Its adhesive and waterproofing properties have been known since early civilization (*1*). It was used by a thriving shipbuilding industry in Samaria about 6000 B.C. An ancient civilization in the Indus Valley (northwestern India) used asphalt cement in the construction of large public baths or tanks about 3000 B.C. They were built by placing a one-inch layer of asphalt cement between two courses of baked brick. As early as 2600 B.C, Egyptians used native asphalts for waterproofing, mummification, and building structures. The use of naturally occurring asphalts as a mortar for building and paving blocks, caulking for ships, and numerous waterproofing applications continued in later years in various parts of the world.

Commercial types of asphalt can broadly be classified into two categories (*1*):

1. Natural Asphalts: These were laid down in geologic strata and occur both as relatively soft asphalt material and also as a hard, friable, black material in veins of rock formations, or impregnated in various limestones, sandstone formations, and the like. Two examples of these formations are the Trinidad Lake deposit on the Island of Trinidad, in Bermudez Lake, Venezuela, and in the extensive "tar sands" throughout western Canada.

   Hundreds of thousands of tons of asphalt were removed from the Trinidad Lake without its showing any appreciable sign of loss (*2*). However in recent years some lowering of the asphalt level has been observed. As the material is removed, pressure deep in the earth forces more of the heavy residue to the surface, where nature continues its refining process. Trinidad asphalt contains about 50 to 57 percent asphalt. The remainder is colloidal clay of volcanic origin, plus some inert organic matter. By the last two decades of the nineteenth century, considerable amounts of asphalt from the Trinidad Lake deposit were used for road building in the United States and elsewhere in the world (*3*).

   The first sheet asphalt pavement using Trinidad Lake asphalt was laid in 1876 on Pennsylvania Avenue in Washington, D.C. Trinidad asphalt has a very low penetration (3-10) and therefore is fluxed back for use in paving in the United States. Normally, it is refined so that the volcanic ash is between 3 and 5 percent.

   The hard variety of natural asphalts such as Gilsonite includes asphaltites which are solid asphalts without impurities (silts, clays, etc.). Gilsonite is a black, brittle and hard asphalt material mostly mined in Utah.

2. Petroleum Asphalts: These are colloidally dispersed hydrocarbons in crude petroleum and are obtained by refining petroleum crudes. Following the development of the refining process in the early 1900s and the popularity of automobiles, large quantities of petroleum asphalt were processed by the oil companies. Gradually these refined asphalts became plentiful and of good quality. This has forced the native (natural) asphalts into a position of relative unimportance (*3*).

Asphalt Refining

The material for this section on asphalt refining has largely been obtained from papers by Corbett (*4*) and Hodgson (*5*).

Almost all paving asphalt cement used today is obtained by processing crude oils. Many refineries in the United States are located near water supplied transport or are supplied by pipelines from marine terminals. Therefore, the world supply of crude oil is significant to the supply of asphalt. A substantial amount of crude oil used in the United States is imported from foreign countries. The main countries or regions from which the heavy asphalt–bearing crude oils are imported are Mexico, Venezuela, Canada, and the Middle East. There is also considerable production of asphalt–bearing heavy crude oils in the United States, primarily in the following areas: Gulf Coast, Southwest, Rocky Mountain, West Coast, and the North Slope of Alaska, as shown in Figure 2–1.

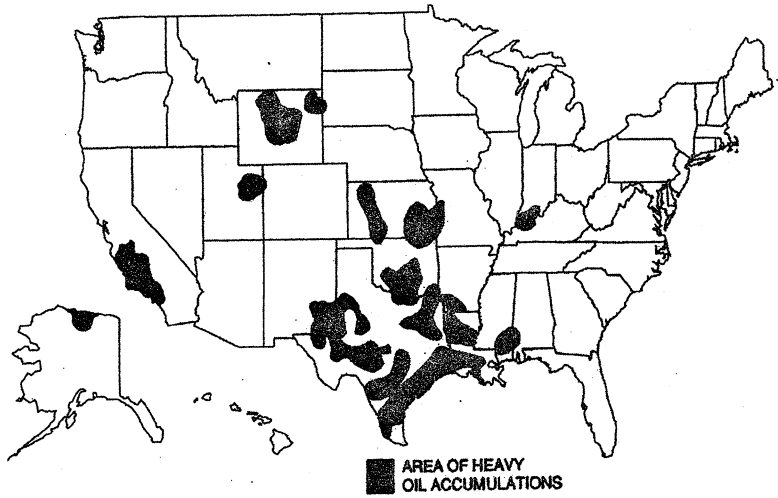


Figure 2–1. Location of Heavy Oil Fields in the United States (after Hodgson, (5)

Crude Petroleum Composition. Crude petroleum varies in composition from source to source. They yield different amounts of residual asphalt cement and other distillable fractions. Figure 2–2 shows typical volume percentages of these fractions in three crudes. The residual asphalt (bitumen) contents are shown by the shaded areas.

Crude oils may be arbitrarily classified according to their API (American Petroleum Institute) gravity, which can be used to estimate the asphalt yield. The API gravity is an arbitrary expression of the density or weight of a unit volume of material expressed at 60°F and is obtained as follows:

$$API = 141.5/Specific\ Gravity - 131.5$$

The API gravity for water is 10. Asphalts have an API gravity of approximately 5–10, whereas the API gravity of gasoline is about 55.

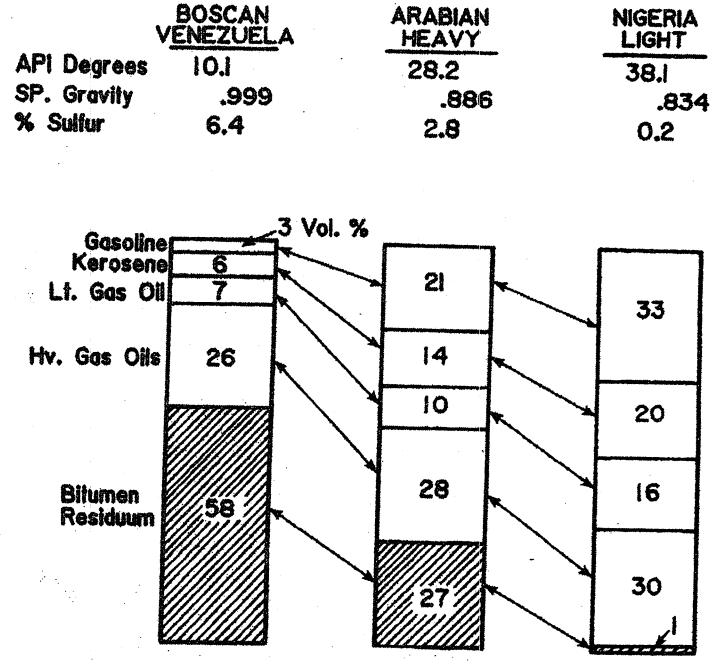


Figure 2-2. Composition of Crude Petroleum (after Corbett, 4)

Low API gravity crudes (API less than about 25) yield relatively low percentages of distillable overhead fractions and high percentages of asphalt cement. On the other hand, the high API gravity crudes (API more than about 25) yield relatively high percentages of overhead fractions and low percentages of asphalt cement. The low API gravity crudes are generally known in the industry as heavy crudes, or as sour crudes if they contain high sulfur content. The high API gravity crudes are referred to as light crudes, or sweet crudes if they contain low sulfur content. Therefore, a refiner must choose the crude type depending on the types and amounts of end product desired to be produced, which may be dictated by the market demand. Each crude is identified by name or source along with its API gravity. Table 2–1 lists some crude types and typical sources.

Table 2–1. Arbitrary Crude Types and Typical Sources (after Corbett, 4)

Type	API	Source
A	34	Arabian Light, Saudi Arabia
	32	Kuwait
B	28	Hawkins, USA
	26	Tiajuana, Venezuela
C	19	Galan, Colombia
	18	Cyrus, Iran
D	16	Lloydminster, Canada
	15	Obeja, Venezuela
E	12	Panuco, Mexico
	10	Boscan, Venezuela

Distillation. The first step in the processing of all crude petroleum is straight reduction by distillation. Figure 2–3 shows the distillation principle which is used to separate various crude fractions which have different boiling ranges. Because asphalt cement is primarily made up of the highest boiling fractions, it becomes the residuum (also called "resid" in the industry) from the vacuum tower. The crude oil is heated in a large furnace to about 650°F and partially vaporized. Then it is introduced into a distillation tower where the lightest components vaporize, rise to the top, cool, condense, and are drawn off for further processing. At various heights in the tower, different fractions reach their boiling point and then, as the temperature is reduced, condense on trays inside the tower. The intermediate or mid–tower components are drawn off at those levels and are treated to make jet fuel, kerosene, and diesel. The residuum from this distillation is usually fed into a vacuum distillation unit where heavier gas oils are produced. The reduced pressure (typically 55 mm Hg, 1.0 psi) in the vacuum tower assists in flashing these oils at lower temperatures to prevent thermal cracking of the asphalt cement. In cracking, asphalt molecules are thermally broken apart. Often steam is added to the bottom of the tower to further reduce the pressure from 50

mm Hg to about 12 mm Hg, 0.24 psi. The bottom fraction from this unit is called vacuum processed, steam refined asphalt cement. The grade of asphalt cement, whether it is measured by penetration, viscosity, or performance graded is controlled by the amount of heavy gas oil removed. All of this is done by a continuous flow operation.

Figure 2–3 shows a typical range of distillation temperatures to obtain different overhead fractions. The figure also shows a cut point 800–1050°F (427-565°C) at which the asphalt residuum is fractionated from the overhead fractions above it in the vacuum tower. The cut point is the atmospheric equivalent vapor temperature to effect this fractionation.

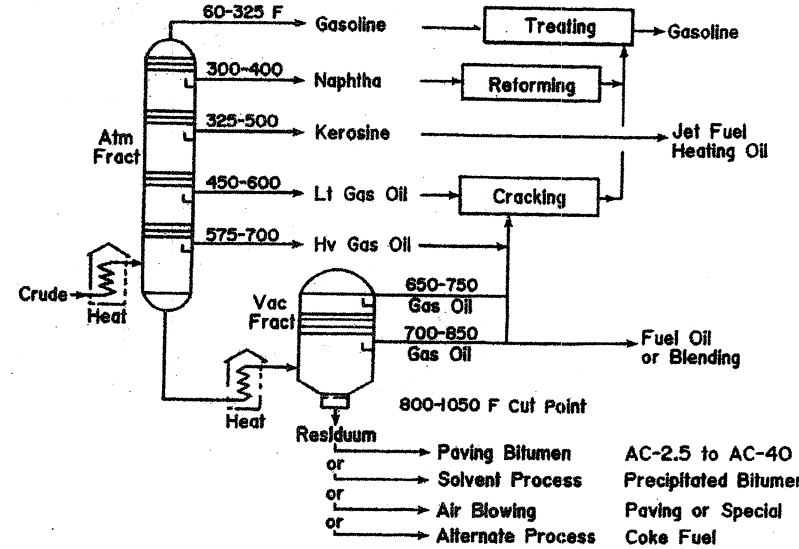


Figure 2–3. Crude Initially Separated by Distillation (after Corbett, 4)

Solvent Deasphalting. A vacuum residuum is subjected to solvent deasphalting (SDA) to extract additional amounts of high boiling fractions (deasphalted oil) for applications such as lube manufacture. Figure 2–4 shows a typical flow process. Extracting solvents generally

used are propane and butane. The vacuum residuum comes in contact, at 100 to 150°F (38 to 66°C), with the solvent in a countercurrent fashion in an extracting (EXT) tower. A high softening point, hard asphalt cement (precipitated asphalt) is obtained in this process. This hard asphalt can be used as a blending component for producing paving grade asphalt cements. An excessive amount of precipitated asphalt component is often detrimental because it increases the temperature susceptibility of the blended asphalt cement.

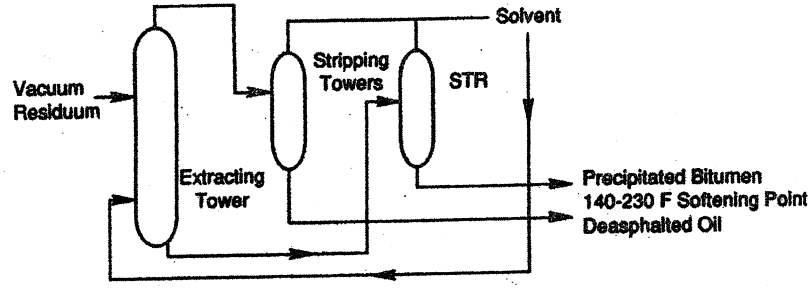


Figure 2–4. Solvent Deasphalting (SDA) (after Corbett, 4)

Solvent Extraction (ROSE). A newer process called the residuum oil supercritical extraction (ROSE) provides a greater flexibility in resid fraction characteristics. The ROSE process is shown in Figure 2–5. A resid feed is mixed with a low-boiling hydrocarbon solvent (for example, normal pentane) under supercritical conditions. This mixture is then fed into a separator (SEP) at a predetermined controlled temperature (T_1) and pressure, to separate the asphaltene concentrate. Asphaltenes (first fraction) are then recovered by stripping (STR) off the solvent used. The extract portion from the preceding operation is then fed into a second separator at a controlled higher temperature (T_2), where this results in a decrease in solubility results, causing the precipitation of resins (second fraction). Oils (third fraction) are then similarly separated and recovered. Asphaltene or resin fraction from the ROSE process is used as a blending component for asphalt cements to meet the specification requirements.

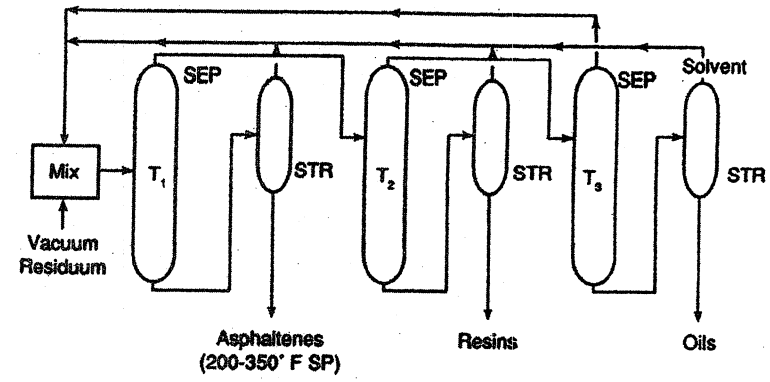


Figure 2–5. Supercritical Extraction (ROSE) (after Corbett, 4)

Continuous Air–Blowing. Continuous air–blowing is normally used when viscosity of vacuum resids must be increased as in producing roofing asphalts, or when the temperature susceptibility of the paving asphalt cement needs to be improved. A schematic for an air–blowing unit is shown in Figure 2–6. The process involves the continuous pumping of a vacuum resid (flux) through a heater where its temperature may be raised to approximately 450 to 500°F (232 to 260°C) into an air–blowing tower. Air is injected at the bottom of the resid and flows upward through the resid. Since the reaction is exothermic (and therefore produces heat), water is usually sprayed onto the surface to cool the asphalt. Under normal circumstances it is potentially hazardous to add water to hot asphalt cement or vice versa because of the tremendous foaming problem. It can be done only in specially designed and controlled processes like this.

Environmental considerations require the gases from the air–blowing operation to be sent to a scrubber where fumes are cooled and the oil is skimmed off. Non–condensable vapors, which may be quite odorous, are sent on to an incinerator. Roofing asphalts are manufactured with the process of batch air–blowing. Continuous air–blowing, which involves a

lesser degree of chemical treatment (oxidation), is normally used for manufacture of paving grade asphalt cement.

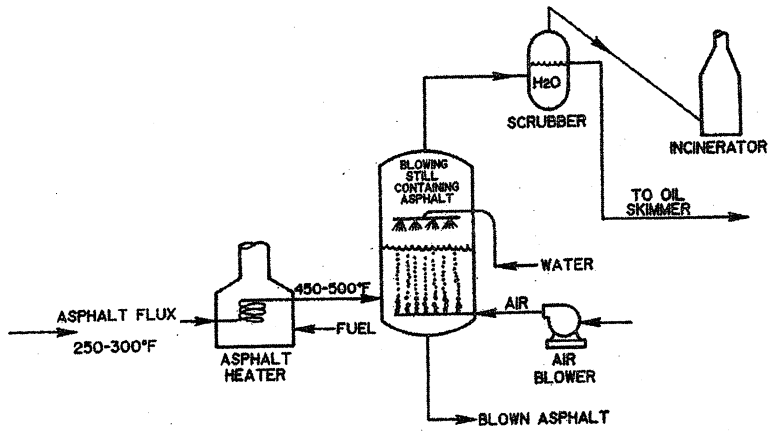


Figure 2–6. Unit for Air–Blowing of Asphalt (after Hodgson, 5)

Coking. Coking is a process that has received more attention in recent years (6, 7, 8). This process allows more of the asphalt cement to be converted into the more expensive materials such as gasoline. So if the price of gasoline increases the refinery with a coking operation can convert more of the asphalt cement to gasoline thus improving the bottom line for the refinery. This process requires that the price of asphalt cement be increased to compete more closely with the price of gasoline since much of the asphalt cement can now be converted to gasoline with the coking process. As a result of the coking operations the price of asphalt does not vary very much from the price of gasoline.

Several Manufacturing Options. Depending on the crude type and available process capabilities, several manufacturing options are available to produce asphalt cements. Although straight reduction to grade is the most direct and most common practice, Figure 2–7 shows other options that may be used. Blending of two or more crude feeds is quite common. If these crudes are refined separately with one yielding a

low viscosity residuum and the other yielding a high viscosity residuum, the resids could be blended in suitable proportions to meet the specification requirements. Also, a low viscosity resid could be blended with a precipitated asphalt or air–blown asphalt to increase the viscosity. A high viscosity resid could be blended back with a gas oil or a similar fraction to reduce the viscosity. However, the selection of a process by the refinery is generally dependent on the type of available crude(s) and the asphalt cement specification requirements. Hard and soft asphalt cements can also be blended in an asphalt terminal to meet a specified grade.

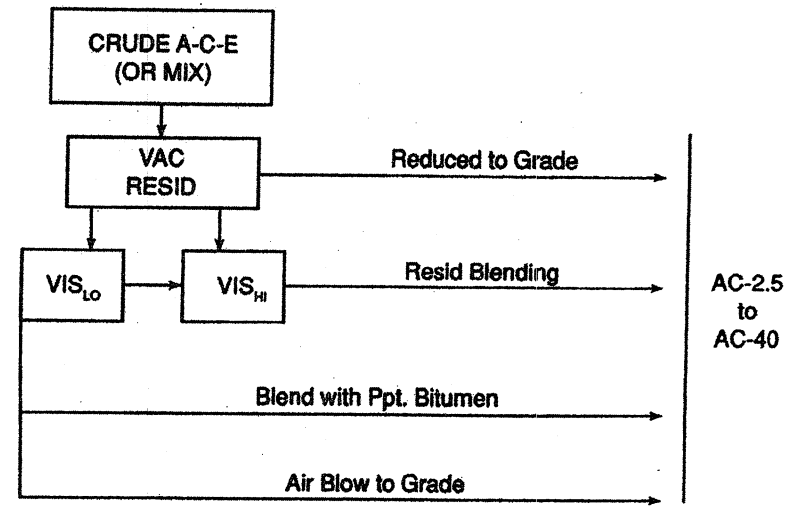


Figure 2–7. Asphalt Cement Prepared by Several Methods (after Corbett, 4)

ASPHALT TYPES AND USES

Asphalts most commonly used in flexible pavement construction can be divided into three types:

1. Asphalt cements;

2. Emulsified asphalts; and
3. Cutback asphalts.

Asphalt Cements

As discussed earlier, asphalt cement is obtained by the distillation process from crude petroleum using different refining techniques. At ambient temperatures asphalt cement is a black, sticky, semisolid, and a highly viscous material. It is a strong and durable cement with excellent adhesive and waterproofing characteristics. It is also highly resistant to the action of most acids, alkalis and salts. The largest use of asphalt cement is in the production of Hot Mix Asphalt (HMA), which is primarily used in the construction of flexible pavements throughout the world. The asphalt cement can readily be liquefied by applying heat for mixing with mineral aggregates to produce HMA. Being very sticky, it adheres to the aggregate particles and binds them together. After compaction and cooling to ambient temperature, HMA is a very strong paving material which can sustain heavy traffic loads such as those experienced on interstate highways and heavy duty airfields. As discussed in Chapter 1, 2.4 million miles of the 2.6 million miles of paved roads (approximately 94%) are covered with some type of asphalt surface.

In the literature, HMA has numerous synonyms. It has been referred to as asphalt concrete, asphaltic concrete, asphalt cement concrete, asphalt mixture, asphalt paving mixture, bituminous concrete, bituminous mixture, bituminous paving mixture, etc. In this textbook an attempt has been made to use the term HMA as much as possible.

Several standard grades of asphalt cement, based on consistency, are available commercially. Three methods have been primarily used to classify asphalt cements: penetration, viscosity, and performance grade. The penetration grading of asphalt cement is specified in ASTM D946 and is primarily controlled by the penetration test. Five standard penetration graded asphalt cements, 40–50, 60–70, 85–100, 120–150, and 200–300, are used for paving grade asphalt. The penetration test and

the penetration grading system will be discussed in detail later in this chapter. It will suffice to mention here that the penetration test involves measurement of penetration with a standard needle into a sample of asphalt cement under standard temperature, time, and load. Obviously, the higher the penetration, the softer the asphalt cement. Therefore, a 40–50 pen grade is the hardest grade, and a 200–300 pen grade is the softest grade.

The second method of classifying asphalt cements is by viscosity grading, as specified in ASTM D3381. This grading is based on either the viscosity of the original asphalt cement or on the viscosity of the asphalt cement after aging in the rolling thin film oven (RTFO) test. Both viscosities are measured at 140°F (60°C) and reported in poises. The viscosity grades based on original asphalt cements include AC–2.5, AC–5, AC–10, AC–20, AC–30, and AC–40. The numerical values indicate viscosity at 140°F (60°C) in hundreds of poises. The viscosity grades based on the asphalt residue (AR) from the RTFO test include AR–1000, AR–2000, AR-4000, AR–8000, and AR–16000. The numerical values indicate viscosity of the aged asphalt at 140°F (60°C) in poises. The viscosity tests and the viscosity grading systems will be discussed in detail later in this chapter.

The third method of classifying asphalt binders is performance based grades which were developed in the Strategic Highway Research Program (SHRP). This classification involves testing the asphalt cement at an upper performance temperature as well as at a lower performance temperature. It involves testing the asphalt unaged, aged in the thin film oven test, and aged in the pressure aging vessel. The primary tests include dynamic shear rheometer, bending beam rheometer, and direct tensile test. It is discussed later under the section titled "Superpave binder tests and specifications." Typical grades of PG graded binders include PG 64-22, PG 64-28, PG 70-22, and PG 76-22. There are many more grades but these provide examples of some of the grades that are available. In essence a PG 64-22 should provide an asphalt cement that is stiff enough to resist rutting up to 64 degrees C and flexible enough to resist thermal cracking down to -22 degrees C.

Emulsified Asphalts

Emulsified asphalt (also simply called emulsion or asphalt emulsion) is a mixture of asphalt cement, water, and emulsifying agent. Because the asphalt cement will not dissolve in water, asphalt cement and water exist in separate phases as shown in Figure 2–8. Hot asphalt cement and water containing the emulsifying agent (soap is one example) are passed under pressure through a colloid mill to produce extremely small (less than 5–10 microns) globules or droplets of asphalt cement which are suspended in water (Figure 2–9). The emulsifying agent imparts an electric charge to the surface of the droplets which causes them to repel one another, and thus the globules do not coalesce. Emulsified asphalts are also categorized as liquid asphalts because, unlike asphalt cements, they are liquid at ambient temperatures. Emulsions are made to reduce the apparent viscosity of the asphalt so that it can be handled at lower application temperatures.

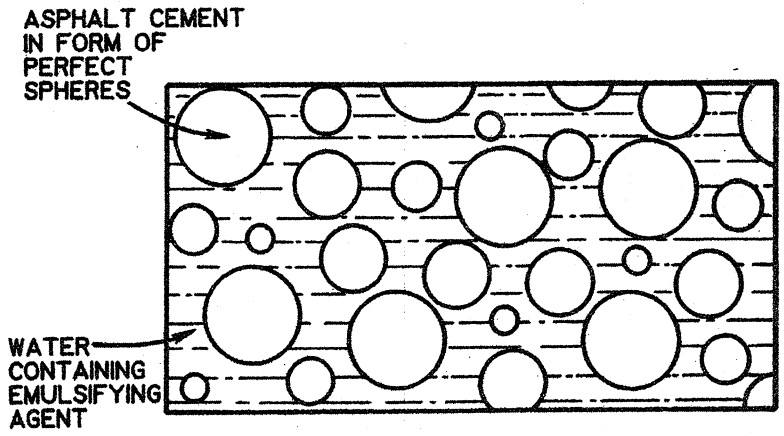


Figure 2–8. Emulsified Asphalt

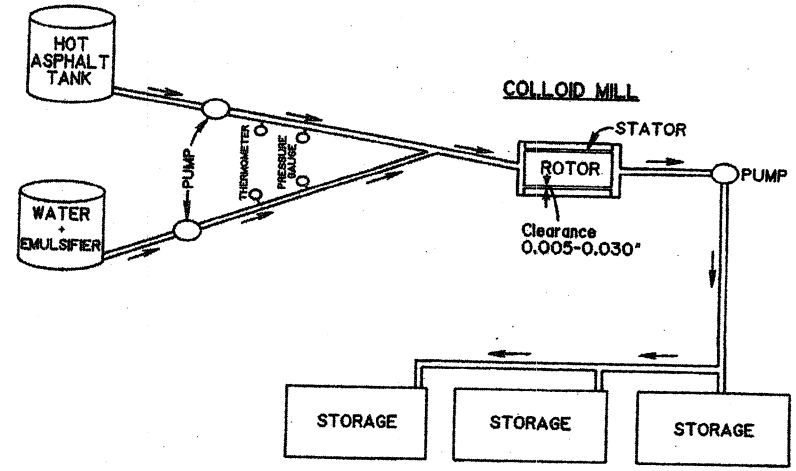


Figure 2–9. Manufacture of Emulsified Asphalt.

The two most commonly used types of emulsified asphalts are specified in ASTM D977 and ASTM D2397:

1. Anionic—electro–negatively charged asphalt droplets; and
2. Cationic—electro–positively charged asphalt droplets.

If the emulsifying agent is anionic, the asphalt droplets bear a negative charge. If the emulsifying agent is cationic, the droplets bear a positive charge.

Most mineral aggregates bear a positive or a negative or mixed charge on the surface. Most siliceous aggregates, such as sandstone, quartz and siliceous gravel, are negatively charged and therefore are generally compatible with the positively charged cationic emulsified asphalts. On the other hand, some aggregates such as limestone bear a positive surface charge and are therefore generally compatible with the negatively charged anionic emulsified asphalts. This happens because opposite charges attract one another.

When an emulsified asphalt is mixed with an aggregate, it "sets" or "breaks" because the asphalt droplets react with the surface of the aggregate and coalesce, squeezing out the water between them. The

evaporation of water is the primary method which finally causes the anionic emulsified asphalt to "break" or "set" and produce a continuous film of asphalt on the aggregate or pavement. Cationic emulsified asphalts break primarily by electro–chemical processes.

Both anionic and cationic emulsified asphalts are further graded according to their "setting" rate. The anionic emulsified asphalts include rapid setting (RS), medium setting (MS), and slow setting (SS) as specified in ASTM D977. The setting rate is controlled by the type and amount of the emulsifying agent. The anionic grades are: RS–1, HFRS–2, RS–2, MS–1, HFMS–2, MS–2, MS–2h, SS–1, and SS–1h. The h designation means that a harder base asphalt cement is used in the emulsion. The HF designation refers to a high float residue, which is an indication of chemical gelling of the emulsion residue.

The cationic emulsified asphalts which are specified in ASTM D2397 include rapid setting (CRS), medium setting (CMS), and slow setting (CSS) grades. The cationic grades are CRS–1, CRS–2, CMS–2, CMS–2h, CSS–1, and CSS–1 h.

Selection and uses of emulsified asphalts are given in ASTM D3628. Generally, they are used as follows:

1. Rapid Setting Grades: Surface treatments, penetration macadams, and tack coat;
2. Medium Setting Grades: Open–graded cold asphalt–aggregate mixtures; and
3. Slow Setting Grades: Tack coat, fog seal, dense–graded cold asphalt–aggregate mixtures, and slurry seals.

Cutback Asphalts

Cutback asphalts are liquid asphalts which are manufactured by adding petroleum solvents (also called cutter stock or diluent) to cut back (make less viscous) the asphalt cements. They are made to reduce the asphalt viscosity for lower application temperatures. Application to aggregate or pavement causes the solvent to escape by evaporation, thus

leaving the asphalt cement residue on the surface. Based on the relative rate of evaporation, cutback asphalts are divided into three types:

1. Rapid Curing (RC)—produced by adding a light diluent of high volatility (generally gasoline or naphtha) to asphalt cement. These are used primarily for tack coat and surface treatments.
2. Medium Curing (MC)—produced by adding a medium diluent of intermediate volatility (generally kerosene) to asphalt cement. These are generally used for prime coat, patching mixtures that can be stockpiled, and road–mixing operations.
3. Slow Curing (SC)—produced by adding oils of low volatility (generally diesel or other gas oils) to asphalt cement. They are also called road oils. They are generally used for prime coat, patching mixtures that can be stockpiled, and as dust palliatives.

Cutback asphalts are commercially available in different grades as shown in Figure 2–10. The thinnest and most fluid grade is designated by the suffix number 30, which is available in MC type only. Other suffix numbers, 70, 250, 800, and 3000, denoting increasingly higher viscosities, are available in all types. These suffix numbers represent the minimum kinematic viscosity in centistokes at 140°F for the particular grade. Specifications for SC, MC, and RC type cutback asphalts are given in ASTM D2026, ASTM D2027, and ASTM D2028, respectively.

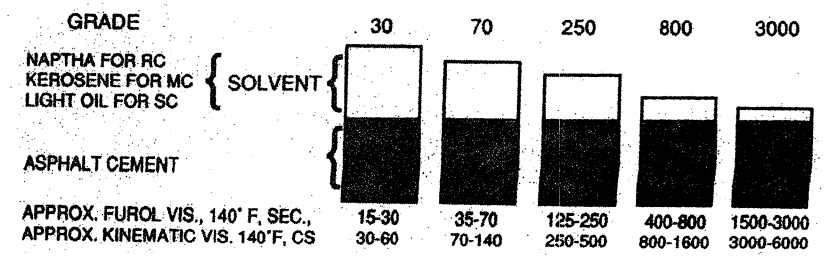


Figure 2–10. Composition of Cutback Asphalts

Emulsified asphalts are increasingly being used in lieu of cutback asphalts for the following reasons:

1. Environmental regulations. Emulsions, unlike cutback asphalts, primarily use water thus resulting in only small amounts of volatiles to evaporate into the atmosphere along with the water.
2. Loss of high energy products. When cutback asphalts cure, the diluents which are high energy, high price products are wasted into the atmosphere.
3. Safety. Emulsions are safe to use. There is little danger of fire as compared to the cutback asphalts, some of which have very low flash points.
4. Lower application temperature. Emulsions can be applied at relatively low temperatures compared to cutback asphalts, thus saving fuel costs. Emulsions can also be applied effectively to a damp pavement, whereas drier conditions are required for cutback asphalts.

ASPHALT CEMENT PHYSICAL TESTS

Physical testing of asphalt cements can be categorized as follows:

1. Consistency tests;
2. Durability tests;
3. Purity tests;
4. Safety tests;
5. Performance tests; and
6. Other tests.

Consistency Tests

Consistency describes the degree of fluidity of asphalt cement at any particular temperature. Since asphalt cement is a thermoplastic material, its consistency varies with the temperature. Therefore, it is necessary to measure the consistency of different asphalt cements at the same temperature and shear loading conditions if comparisons are to be made. These tests are not normally used much today since the adoption of

Superpave binder tests but they are presented here for historical purposes. The following consistency tests have been used to evaluate asphalt cements:

Absolute Viscosity at 140°F (60°C). Viscosity can simply be defined as resistance to flow of a fluid. Viscosity grading of asphalt cements is based on viscosity measurement at 140°F (60°C). This temperature was selected because it approximates the maximum HMA pavement surface temperature during summer in the United States.

A capillary tube viscometer is used to perform the viscosity test at 140°F (60°C). Two viscometers, the Cannon–Manning vacuum viscometer (Figure 2–11) and the Asphalt Institute vacuum viscometer (Figure 2–12), are commonly used. ASTM D2171 test method describes the test procedures. The viscometer is mounted in a thermostatically controlled, constant temperature water or oil bath which is maintained at 140°F (60°C). The viscometer tube is charged with asphalt cement through the large side until the level of asphalt cement reaches the filling line. After the filled viscometer tube is kept in the bath for a prescribed period of time to obtain the equilibrium temperature of 140°F (60°C), a partial vacuum is applied to the small side of the viscometer tube to cause the asphalt cement to flow. Application of partial vacuum is necessary because the asphalt cement is too viscous to flow at this temperature. A vacuum control device and a vacuum pump are needed as part of the testing equipment. After the asphalt cement starts to flow, the time (in seconds) required for it to flow between two timing marks is measured. The measured time (in seconds) is multiplied by the calibration factor for the viscometer tube to obtain the value for viscosity in poises, which is the standard unit for measuring viscosity. Manufacturers calibrate their viscometer tubes with standard oils and furnish the calibration factor with each tube.

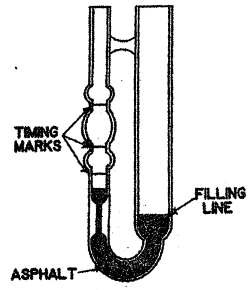


Figure 2–11. Cannon–Manning Vacuum Viscometer

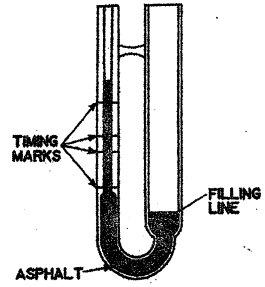


Figure 2–12. The Asphalt Institute Vacuum Viscometer

When testing a material such as asphalt cement of an unknown viscosity, the shear stress is the same as that when calibrating the viscometer tube. The only difference is the measured rate of shear, which is inversely proportional to the time required to pass a liquid through the tube. The following relationship can be used to determine the viscosity:

$$\frac{V_2}{T_2} = \frac{V_1}{T_1}$$

or

$$V_2 = T_2\left(\frac{V_1}{T_1}\right)$$

where,

V_1 = viscosity of standard material;

T_1 = time for standard material to pass through the tube;

V_2 = viscosity of unknown material;

T_2 = time for unknown material to pass through the same tube; and

$\frac{V_1}{T_1}$ = calibration factor for tube.

Kinematic Viscosity at 275°F (135°C). ASTM D2170 is used to measure the kinematic viscosity at 275°F (135°C) using a Zeitfuchs Cross–Arm viscometer (Figure 2–13). At this temperature the asphalt cement is sufficiently fluid to flow through the capillary tube under gravitational forces alone, and there is no need to apply any partial vacuum. This temperature was selected because it approximates the mixing and laydown temperatures used in the construction of HMA pavements.

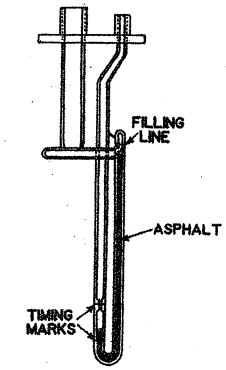


Figure 2-13. Zeitfuchs Cross–Arm Viscometer

Asphalt cement is poured into the large opening of the viscometer until it reaches the filling line. The viscometer is then placed in a clear oil bath medium to obtain the equilibrium temperature. A slight vacuum is applied to the small opening or a slight pressure is applied to the large opening to induce the flow of the asphalt cement over the siphon section just above the filling line. Then the gravitational forces cause the asphalt

cement to flow downward in the vertical section of the capillary tube. A stopwatch is used to measure the time (in seconds) required for the asphalt cement to flow between the two timing marks. The kinematic viscosity in centistokes is obtained by multiplying this time by the calibration factor supplied with the viscometer tube. The unit of centistokes is used in kinematic viscosity at 275°F (135°C) because gravitational forces induce flow and the density of the material affects the rate of flow. Absolute viscosity (such as obtained at 140°F or 60°C) can be calculated from kinematic viscosity by multiplying it by the density of asphalt cement.

Example 2–1: An asphalt cement has a kinematic viscosity of 800 centistokes. Its specific gravity is 1.03. What is its absolute viscosity in poises?

1 stoke = 100 centistokes

Absolute viscosity in poises = kinematic viscosity in stokes x specific gravity
= 8 x 1.03
= 8.24 poises

Penetration. The penetration test is an empirical test used to measure the consistency of asphalt cement. Usually penetration is measured at 77°F (25°C) which also approximates average service temperature of the HMA pavements. At this temperature no simple test method is currently available to measure the consistency of asphalt cement in terms of viscosity, although viscosity is the preferred form of measurement, so the penetration test is used. The standard penetration test is shown in Figure 2–14. A container of asphalt cement is brought to the standard test temperature (usually 77°F or 25°C) in a thermostatically–controlled water bath. The sample is placed under a needle of prescribed dimensions. The needle is loaded with a 100 g weight and is allowed to penetrate the asphalt cement sample for 5

seconds. The depth of penetration is measured in units of 0.1 mm (dmm) and is reported as penetration units. For example, if the needle penetrates 8 mm, the penetration of asphalt cement is 80.

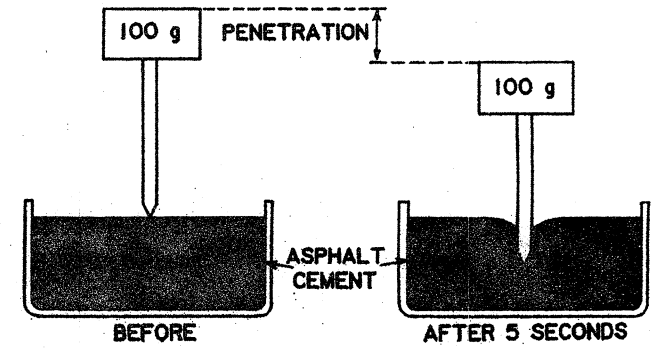


Figure 2–14. Schematic of Penetration Test

The penetration test is run in accordance with ASTM D5. The test can also be run at other temperatures for example 39.2°F (4°C). However, the needle load or penetration time or both are sometimes varied when running the test at other temperatures. At low temperatures, such as 39.2°F (4°C), the asphalt cement is very stiff and therefore penetration of the needle is significantly lower if it is loaded with a 100 g weight and is allowed to penetrate for 5 seconds. At these temperatures, a 200 g weight and 60 seconds penetration time have been used to obtain penetration values.

Softening Point. Softening point is measured by the ring and ball (R & B) method in accordance with ASTM D36. It can simply be defined as the temperature at which an asphalt cement cannot support the weight of a steel ball and starts flowing. Its purpose is to determine the temperature at which a phase change occurs in the asphalt cement. Although its use in specifying paving asphalt cement is quite common in Europe, it is mostly used in the United States for high viscosity roofing asphalts.

As shown in Figure 2–15, the test consists of taking a brass ring filled with asphalt cement and suspending it in a beaker filled with water or ethylene glycol. A steel ball of specified dimensions and weight is placed in the center of the sample. The bath is heated at a controlled rate of 5°C/minute. When the asphalt cement softens, the ball and asphalt cement sink toward the bottom of the beaker. The temperature is recorded at the instant when the softened asphalt cement sinks the prescribed distance and touches the bottom plate. The test is conducted with duplicate specimens. If the difference between the two temperatures exceeds 2°F, the test must be repeated.

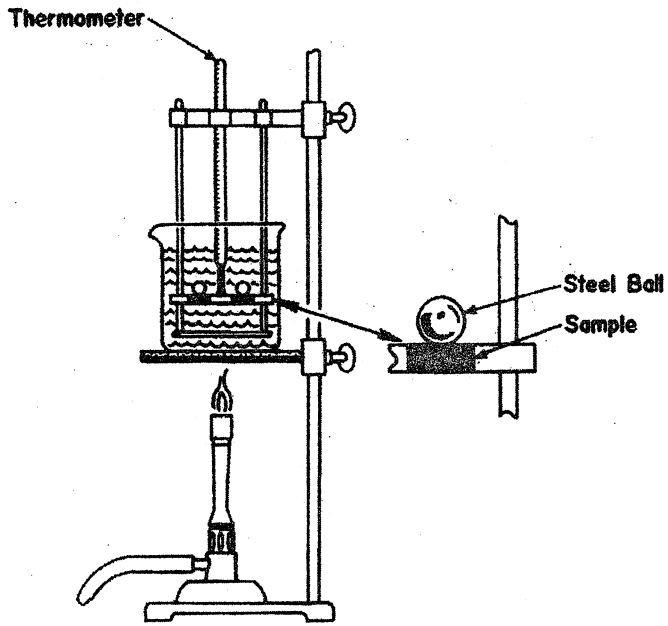


Figure 2–15. Softening Point Test

Ductility Test. Many asphalt paving technologists consider ductility as an important property of asphalt cement. Its significance will be discussed later in this chapter. The ductility test, run in accordance with ASTM D113, measures the distance in centimeters that a standard

briquette of asphalt cement will stretch before breaking (Figure 2–16). The cross section of the briquette at its smaller dimension is one square centimeter. The test sample is brought to temperature in a water bath which is maintained at the standard temperature of 77°F (25°C). The two ends of the sample are pulled apart at the rate of 5 cm/minute until rupture. The water must be at the same specific gravity as the asphalt to prevent floating or sinking of the stretched sample. Salt is added to increase the specific gravity, and alcohol is added to decrease the specific gravity of the water. Some engineers believe in running the ductility test at lower temperatures such as 60 and 39.2°F (16 and 4°C) rather than 77°F (25°C). The pulling rate is usually lowered to 1 cm/minute when the test is run at 39.2°F (4°C) to allow for more stretching prior to break.

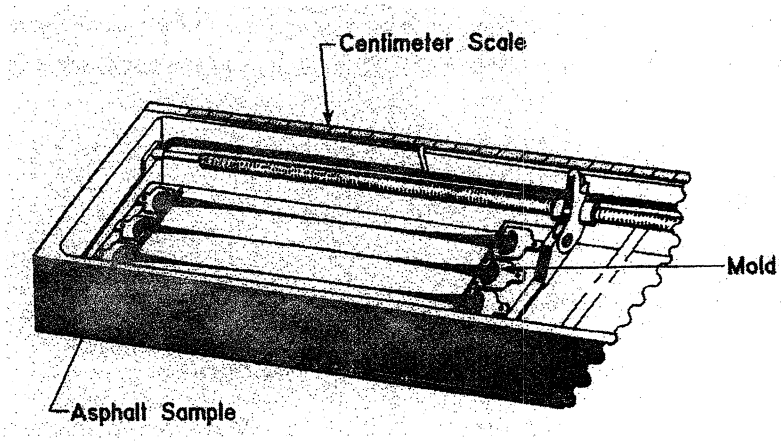


Figure 2–16. Ductility Test

Durability Tests

Asphalt cements undergo substantial short–term aging (hardening) when they are mixed with hot mineral aggregates in a HMA mixing plant. Their long–term aging continues during the life of HMA pavements which are subjected to environmental and other factors discussed later in this chapter. The thin film oven test and the rolling thin

film oven test are generally used to approximate the short–term hardening conditions which occur in normal HMA mixing plants.

Thin Film Oven Test. The thin film oven (TFO) test is conducted by placing a 50 g sample of asphalt cement in a cylindrical flat–bottom pan (5.5 inches or 140 mm inside diameter and 3/8 inch or 9.5 mm deep). The asphalt cement layer in the pan is about 1/8 inch (3 mm) deep. The pan containing the asphalt cement sample is transferred to a shelf in a ventilated oven maintained at 325°F or 163°C (Figure 2–17). The shelf rotates at 5 to 6 revolutions per minute. The sample is kept in the oven for 5 hours, and then transferred to a suitable container for measuring penetration or viscosity of the aged asphalt cement. The test method is described in ASTM D1754. The aged asphalt cement is usually required to meet specified minimum percentage retained penetration or to have some maximum viscosity. A loss or gain in weight of the test sample is also measured and reported.

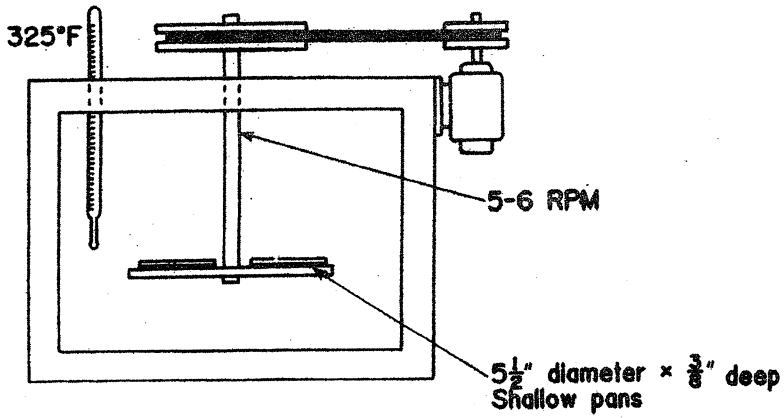


Figure 2–17. Thin Film Oven Test

Rolling Thin Film Oven Test. A variation of the thin film oven test, rolling thin film oven test, is in use by many agencies, although it has the same purpose. Figure 2–18 shows the rolling thin film oven (RTFO) used

in ASTM D2872. A specified amount of asphalt cement is poured into a bottle, which is placed in a rack in the oven maintained at 325°F (163°C). The rack rotates at a prescribed rate around a horizontal axis. The rotating bottle continuously exposes fresh asphalt. The orifice of the sample bottle passes in front of an air jet during each rotation. The vapors accumulated in the sample bottle are purged by the heated air from the jet.

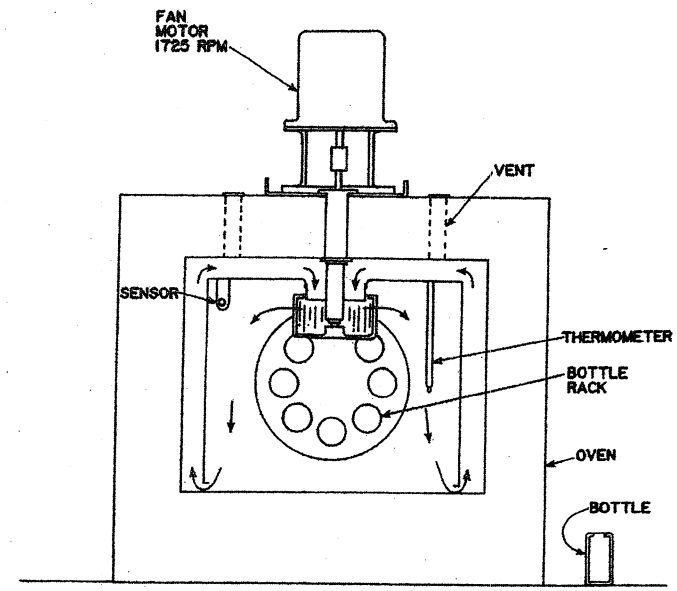


Figure 2–18. Rolling Thin Film Oven Test

The RTFO achieves about the same degree of hardening (aging) as the TFO but in less time (only 75 minutes). Also, it can accommodate a larger number of samples than the TFO.

Purity Tests

Refined asphalt cements consist of almost pure bitumen which, by definition, is entirely soluble in carbon disulfide. Only very little

amounts of impurities are generally present in refined asphalt cements. To determine the purity of asphalt cement, a solubility test (ASTM D2042) is conducted. An asphalt cement sample of known weight is dissolved in a solvent and is then filtered through a glass fiber pad. The insoluble material retained on the pad is washed, dried and weighed. The insoluble material constitutes the impurity in the asphalt cement sample. Specifications for paving grade asphalt cements generally require a minimum of 99.0 percent solubility. It is also desirable that the asphalt cement be free of moisture because it can cause the asphalt cement to foam when it is heated above 212°F (100°C).

Safety Tests

If asphalt cement is heated to a high enough temperature, it gives off enough vapor to flash (ignite) in the presence of a spark or open flame. Flash point indicates the temperature to which asphalt cement may be safely heated without the danger of instantaneous flash in the presence of an open flame. This temperature is below that of the fire point, which is the temperature where a material combusts. Although the flash point of paving asphalt cements is well above the temperatures normally used in HMA production, it is necessary to measure and control it for safety considerations.

The Cleveland Open Cup method (ASTM D92) is generally used for determining the flash point of paving asphalt cements (Figure 2–19). A brass cup partially filled with asphalt cement is heated at a prescribed rate. A small flame is passed over the surface of the cup periodically. The temperature at which sufficient vapors are released to produce an instantaneous flash is reported as the flash point.

Performance Tests

With the new Performance Graded system there are a number of performance tests that are included as part of the system. Three of the performance tests include dynamic shear rheometer, bending beam

rheometer, and direct tension test. These tests are discussed in more detail under the Asphalt Cement Performance Grading Systems.

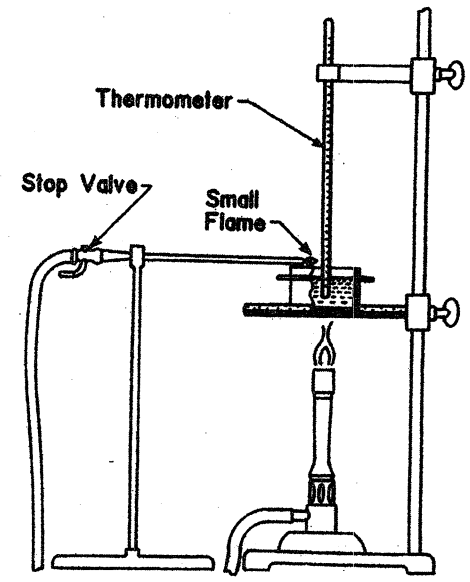


Figure 2–19. Cleveland Open Cup Flash Point Test

Other Tests

Specific Gravity. Specific gravity is defined as the ratio of the mass of the material at a given temperature to the mass of an equal volume of water at the same temperature. Specific gravity of the asphalt cement changes when the asphalt cement expands on heating. Therefore, specific gravity determinations are useful in making temperature–volume corrections or determining the weight per unit volume of asphalt cement heated to its application temperature.

The pycnometer method is used to determine the specific gravity of asphalt cements (ASTM D70). Since the specific gravity varies with the temperature, test results are expressed in terms of specific gravity (Sp. Gr.) at a given temperature for both the asphalt cement and the water

used in the test. For example, Sp. Gr. 1.02 at 60°/ 60°F (15.6°/15.6°C) means that the asphalt cement specific gravity is 1.02 when both the asphalt cement and the water are at 60°F (15.6°C). Specific gravity at 60°/60°F (15.6°/15.6°C) is normally used for making monetary compensations or transactions concerning asphalt cements.

Spot Test. The purpose of the spot test is to determine whether an asphalt cement has been damaged during processing due to overheating, resulting in the eventual cracking of the asphalt cement. When asphalt cement cracks, the molecules are thermally broken apart. According to some engineers, the asphalt cement thus produced is inferior and is more susceptible to weathering or aging. Since cracking is not likely to occur in the current refining practices, the spot test is usually not a requirement in the specifications. Also, cracked asphalt cements tend to have poor ductility test results and high age hardening rates.

This test, developed by Oliensis, a chemist, is a crude form of paper chromatography in that it involves a visual evaluation of a spot of asphalt cement dissolved in a standard solvent (such as naphtha). A drop of the asphalt/solvent mixture is placed on a special filter paper and the results observed visually. If the spot formed is a uniformly brown stain, the test is reported as negative and the asphalt cement is acceptable. However, if, within the spot, there is a darker area at the center, the test is reported as positive and will disqualify the asphalt cement. The significance of the spot test is still in dispute. The spot test cannot be applied to an asphalt cement which has been extracted and recovered from HMA.

ASPHALT CEMENT GRADING SYSTEM

Historical Background

Halstead and Welborn have given a very good account of the early history of the development of asphalt specifications in Reference 9. This account was prepared before the performance graded (PG) grading system was developed. Even though most everyone in the United States

and many places outside the United States are now using the PG grading system, there are still some places that use the viscosity or penetration grading system. This discussion of penetration and viscosity grading is also provided for historical background.

From 1878 to 1900 most of the asphalt cement used in the construction of sheet asphalt, penetration macadam, and mixed macadam (now called hot mix asphalt) was obtained from the Trinidad Lake. The Barber Asphalt Paving Company was the primary supplier of Trinidad Asphalt and builder of asphalt pavements beginning in 1878. Bermudez Lake asphalt found in Venezuela came into use during the 1890s and was a major source by 1901. Like the Trinidad Lake asphalt, the Bermudez asphalt had to be mined, and water and extraneous inorganic and organic matter had to be removed. Both Lake asphalts were found to have remarkable uniformity.

The first specification for asphalt cement in the United States was based on the appearance of the crude Trinidad asphalt and on analytical tests to determine amounts of bitumen (soluble in carbon disulfide) and insoluble organic and inorganic matter. Such specifications were devised merely to identify the source of asphalt cement at the exclusion of other source materials. As the HMA industry grew and petroleum asphalts came into use in the United States about 1900, it became evident that the element of uncertainty in material requirements, especially the consistency, had to be removed. The native asphalts were solid or semi–solid materials, and it was necessary to add fluxing materials to reduce their consistency for use as binders in HMA mixtures. Standardized methods were needed to determine the consistency of the paving binders obtained from different sources.

In 1888, H.C. Bowen invented the Bowen Penetration Machine, the forerunner of the penetrometer, to determine consistency of the asphalt cement. Previous to Bowen's invention, the method of testing the proper degree of softening of the asphalt cement was by chewing. Even after the invention of the Penetration Machine, the chewing method, crude as it may now appear, served as a valuable check. An asphalt man generally

prided himself on the fact that by chewing he could achieve approximately the same results as those obtained by the machine (*9*).

After several modifications of the initial Penetration Machine, by 1910 the penetrometer became the principal means of measuring and controlling consistency of semi–solid asphalts at 77°F (25°C). The Bureau of Public Roads (now the Federal Highway Administration), since 1901, and the American Society for Testing and Materials (ASTM), since 1903, have been instrumental in making the penetration test a standard for controlling the consistency of paving asphalt cements.

In 1918, the Bureau of Public Roads introduced the penetration grading system by developing various penetration grades suited to different climatic conditions and applications. The American Association of State Highway Officials (AASHO) published the standard specifications for penetration graded asphalt cements in 1931.

The next major change in asphalt cement grading specifications was initiated in the early 1960s by the FHWA, ASTM, AASHTO, industry, and a number of state highway departments that wanted the asphalt cements to be graded by viscosity at 140°F (60°C). The primary objectives were: (a) to replace the empirical penetration test with a rational scientific viscosity test, and (b) to measure the consistency at 140°F or 60°C (rather than 77°F or 25°C), which approximates the asphalt pavement maximum surface temperature on a hot summer day in most of the United States. Various viscosity grades were developed to suit the different climatic conditions and applications.

During the early 1960s when the viscosity grading system was being developed, the California Department of Highways was developing a parallel aged residue (AR) viscosity grading with the cooperation of the Pacific Coast User Producer Group. AR viscosity grading was based on the viscosity of the aged residue resulting from the rolling thin film oven (RTFO) test rather than the viscosity of the original asphalt cement as recommended by the FHWA. The California Department of Highways had experienced mix setting problems (tender mix) with some asphalt cements whose original viscosity did not increase during the plant mixing phase as much as that of other asphalt cements. Therefore, they

wanted to grade the asphalt after aging in the RTFO so that all asphalts after mixing would behave about the same during construction.

Figure 2–20 shows the grading systems used in the U.S. during 1985 (*10*). Several states were using more than one grading system. A detailed discussion of the various grading systems follows. The performance based asphalt binder grading system developed in the Strategic Highway Research Program (SHRP) is now used in most all states and will be presented later.

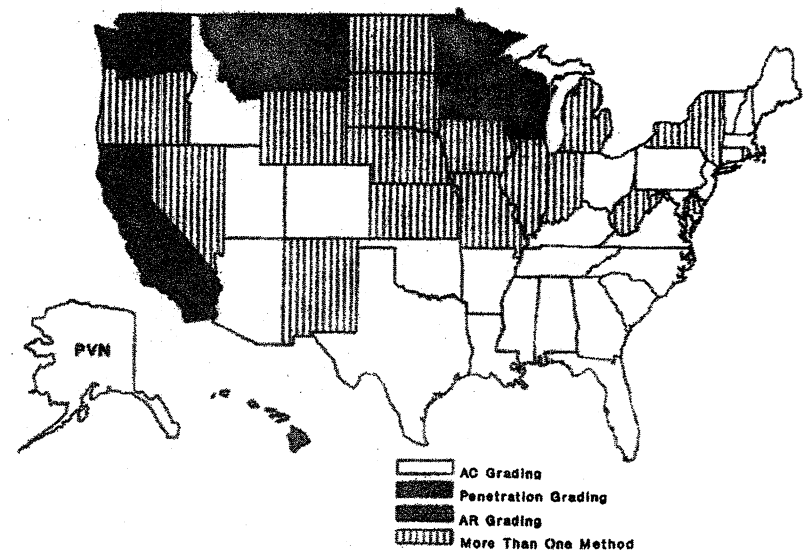


Figure 2–20. Grading Methods for Asphalt Cements Used During 1985 (after Terrel et al, *10*)

As mentioned earlier penetration grades are specified in ASTM D946: 40–50, 60–70, 85–100, 120–150, and 200–300 pen. Two grades, 60–70 and 85-100, have been the most commonly used for paving in the U.S. Other requirements, such as ductility at 77°F (25°C) and retained penetration after thin film oven (TFO) test, are also provided in the standard.

The penetration grading system had the following advantages when compared to the viscosity grading system:

1. Grading is based on the consistency of the asphalt cement at 77°F (25°C), which is close to the average pavement service temperature. Testing at 77°F (25°C) may provide a better correlation with low temperature properties than the viscosity test which is measured at 140°F (60°C).
2. Testing time is relatively short.
3. Penetration testing is adaptable to field conditions. It can be used by the HMA contractor to detect contamination of the asphalt cement (usually from other petroleum distillates or asphalt cutbacks in the transport units).
4. Equipment costs are relatively low.
5. Precision limits for the penetration test are well established.
6. Temperature susceptibility (change in asphalt cement consistency with corresponding change in temperature) of the asphalt cement can be determined by measuring penetration at temperatures other than 77°F (25°C). Temperature susceptibility is discussed in detail later.

However, the penetration grading system has the following disadvantages when compared to viscosity grading:

1. Penetration is an empirical test and does not measure the consistency of asphalt cement in fundamental units such as viscosity.
2. Shear rate is high during the test.
3. Shear rate is variable because it depends on the consistency of the asphalt cement.
4. Similitude at 77°F (25°C) can be deceptive to performance at higher and lower service temperatures (Figure 2–21).
5. No viscosity is available to establish mixing and compaction temperatures.

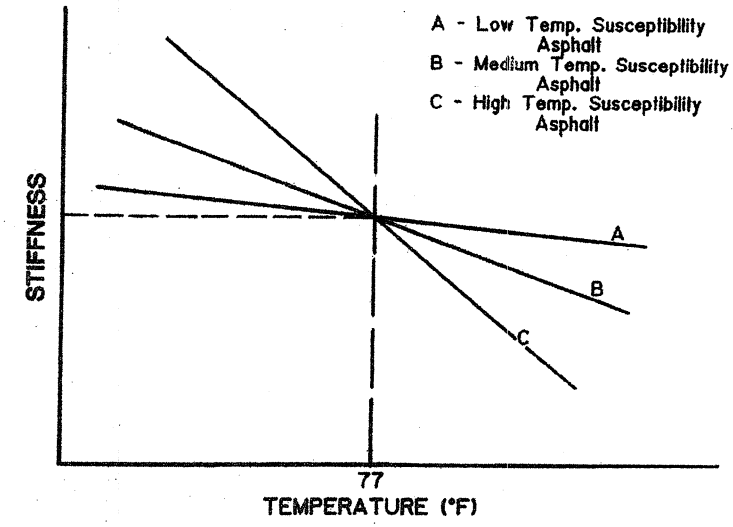


Figure 2–21. Temperature Susceptibility of Penetration Graded Asphalt Cements

Viscosity Grading System

This system, which is based on the viscosity of original (as supplied) asphalt cement, was the most widely used in the United States prior to adopting the performance graded asphalt cement. ASTM D3381 gives the requirements for asphalt cement viscosity graded at 140°F. ASTM Tables 1 and 2 pertain to the following viscosity graded asphalt cements: AC–2.5, AC–5, AC–10, AC–20, AC–30, and AC–40. ASTM Table 3 pertains to the AR (aged residue) viscosity grading system, which is discussed later. AASHTO specification M226 on viscosity graded asphalt cements is similar to ASTM D3381 with minor differences.

The poise is the standard unit of measurement for viscosity. The lower the number of poises, the less viscous the asphalt cement. AC–2.5 (asphalt cement with a viscosity of 250 poises at 140°F or 60°C) is "softer" than AC–40 (asphalt cement with a viscosity of 4000 poises at the same temperature). AC–20 grade is most common viscosity grade

that has been used for paving in the United States. AC–30 grade was primarily used by southeastern states to provide an asphalt a little stiffer than an AC-20 but not as stiff as an AC-40. Other test requirements such as penetration at 77°F (25°C), viscosity at 275°F (135°C), viscosity at 140°F (60°C) and ductility at 77°F (25°C) of the residue from thin film oven test (TFOT), and flash point are also given in the specifications. The penetration at 77°F (25°C) controls the consistency of the asphalt cement near the average service temperature, and the viscosity at 275°F (135°C) controls its consistency near the mixing and compacting temperatures. Together these specified requirements also control the temperature susceptibility of the asphalt cement as shown in Figure 2–22. The primary difference between ASTM Tables 1 and 2 is that the asphalt cements in Table 2 are less temperature susceptible because the specified minimum penetration values at 77°F (25°C) and the minimum viscosity values at 275°F (135°C) are higher than those in Table 1.

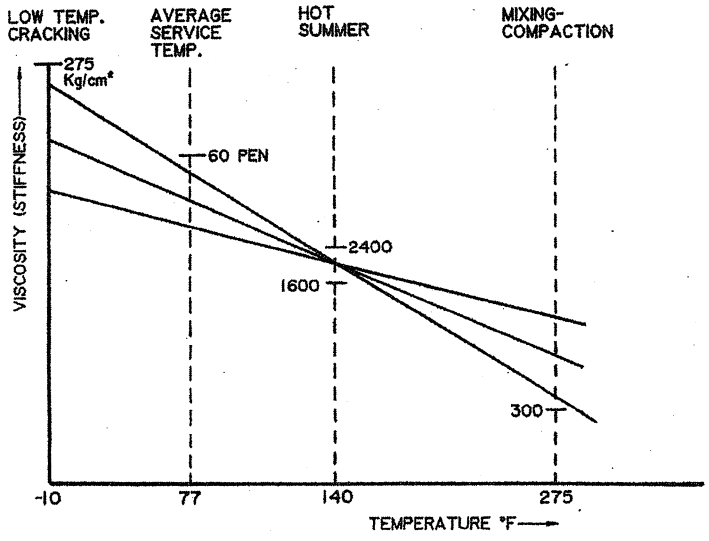


Figure 2-22. Temperature Susceptibility of Viscosity Graded Asphalt Cements

The viscosity grading system has the following advantages over the penetration grading system:

1. Viscosity is a fundamental property, rather than an empirical test and therefore is independent of the test system and the sample size.
2. It is suitable to a wide range of environments (pavement temperatures 77 to 140°F or 25 to 60°C).
3. It is based on viscosity at 140°F (60°C) which is near the maximum pavement surface temperature generally experienced in the United States. This temperature is critical to control pavement performance such as rutting during hot summer days.
4. There is a reduced overlap with other grading systems (Figure 2–23).
5. A wide range of test instruments is available.
6. Test standards are available with established precision limits.
7. Temperature susceptibility of the asphalt cement can be determined since the consistency is measured at three temperatures.

The disadvantages of the viscosity graded system, in comparison to the penetration grading system, are listed below:

1. Grading at 140°F (60°C) is deceptive to performance at average or low service temperatures (Figure 2–22).
2. It is not adequate to safeguard against low temperature cracking.
3. The test system is slightly more expensive than the penetrometer.
4. Testing time is longer.
5. TFOT residue viscosity can vary considerably within the same grade. For example, AC–20 asphalt cements from two different sources can have TFOT residue viscosities of 3,500 and 10,000 poises. These asphalts are likely to behave differently during and after construction.

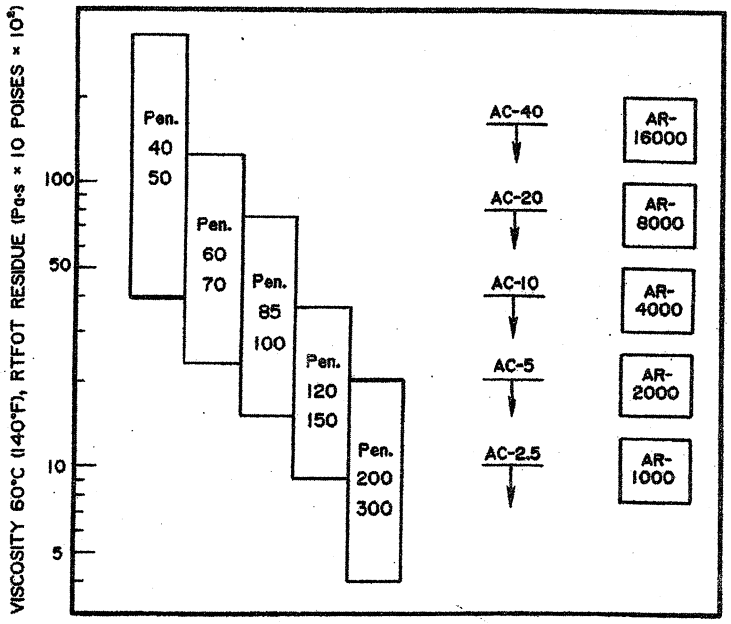


Figure 2–23. Comparison of Penetration Grades and Viscosity Grades of Asphalt Cement (based on RTFOT Residue for AR–Grades and Penetration Grades; TFOT Residue for AC–Grades)

AR Viscosity Grading System

The AR viscosity grading system, which is based on the viscosity of the aged residue (AR) from the rolling thin film oven (RTFO) test, has been used by some states in the western United States. ASTM D 3381 Table 3 gives the requirements for the following five AR grades: AR–1000, AR–2000, AR–4000, AR–8000, and AR–16000. AR–1000 represents a viscosity of 1000 poises at 140°F (60°C) of the aged residue. Specifications also require minimum penetration at 77°F (25°C) and minimum viscosity at 275°F (135°C), again for the aged residue. Penetration of the original asphalt cement (although not specified) is determined to check the specified minimum requirement for percentage of retained penetration after RTFO test. It should be noted that no

consistency values at any temperature are specified for original asphalt cements.

The AR viscosity grading system has the following advantages when compared to the AC grading system:

1. It represents asphalt properties after the HMA is manufactured in a typical HMA batch facility.
2. Reasonable uniform behavior of asphalt cement is expected from different sources but within the same AR grade during laydown and compaction.

The following are the disadvantages of the AR grading system in comparison to the AC grading system:

1. It is highly regional.
2. It requires more testing equipment (RTFO).
3. Testing time is longer because aged residue from the RTFO test must be obtained first.
4. Detecting contamination is a problem because there are no consistency requirements on original asphalt cement.
5. Aged residue may not relate to hardening that occurs in the drum mix process.

RHEOLOGICAL PROPERTIES AND PAVEMENT PERFORMANCE

Rheological Properties

It has been well established that the rheological properties of the asphalt cement binder affect the pavement performance. Since the rheological properties of asphalt cements change during HMA production and continue to change subsequently in service, the phenomenon of aging (or age hardening) is discussed first.

Age Hardening. The first significant hardening of the asphalt cement takes place in the pugmill or drum mixer where heated aggregate is mixed with hot asphalt cement. During the short mixing time, the asphalt cement, which is in very thin films, is exposed to air at temperatures which range from 275 to 325°F (135 to 163°C) or more. Substantial rheological changes such as a decrease in penetration and an increase in viscosity of the asphalt cement take place during this short mixing period from both air oxidation and loss of more volatile components. Age hardening of the asphalt cement continues, although at a much slower rate, while the HMA is processed through a surge or storage silo, transported to the paving site, laid, and compacted. After the HMA pavement has cooled and been opened to traffic, the age hardening process continues at a significantly slower rate for the first 2–3 years until the pavement approaches its limiting density under traffic. Thereafter, the rate of age hardening is further reduced and longer time periods are needed to discern the changes in the rheological properties of the asphalt cement. Age hardening in service takes place at an accelerated rate if the HMA pavement has a higher air void content than originally designed, which provides for easy entry of air, water and light. The rate of hardening is also increased significantly as the temperature of the mixture is increased. Thicker asphalt cement films around the aggregate particles harden at a slower rate compared to thin films. Figure 2–24 shows a typical plot of changing viscosity at 140°F versus time for six asphalt cements used in a durability study (*11*).

The following six factors (not necessarily in order of importance) have been reported to contribute to the age hardening of asphalt cement during mixing and/or in service (*12*, *13*):

1. Oxidation;
2. Volatilization;
3. Polymerization;
4. Thixotropy;
5. Syneresis; and
6. Separation.

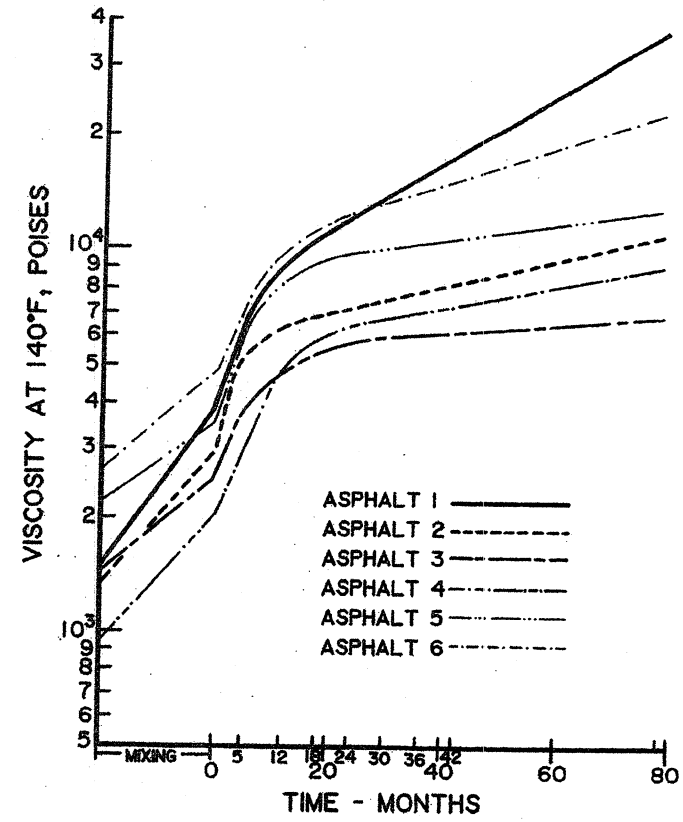


Figure 2–24. Viscosity at 140°F Versus Time in Months (after Kandhal et al, *11*)

These factors have been defined as follows (*12*):

1. Oxidation is the reaction of oxygen with asphalt cement, the rate depending on the character of the asphalt cement and the temperature.
2. Volatilization is the evaporation of the lighter constituents from asphalt cement and is primarily a function of temperature. It is

usually not a significant factor contributing to long–term aging in the pavement.

3. Polymerization is a combining of like molecules to form larger molecules, causing a progressive hardening. There is no scientific evidence that this is a significant factor during the low temperature aging of asphalt in pavements in spite of such speculation in the literature.
4. Thixotropy is a progressive hardening due to the formation of a structure within the asphalt cement over a period of time, which can be destroyed to a degree by reheating and working the material. Thixotropic hardening (also called steric hardening) is generally associated with pavements which have little or no traffic, and its magnitude is a function of asphalt composition.
5. Syneresis is an exudation reaction in which the thin oily liquids are exuded to the surface of the asphalt cement film. With the elimination of these oily constituents, the asphalt cement becomes harder.
6. Separation is the removal of the oily constituents, resins, or asphaltenes from the asphalt cement as caused by selective absorption of some porous aggregates.

Traxler (*14*) has suggested nine additional factors, including several effects of light, water, chemical reaction with aggregate, microbiological deterioration, and adsorption of heavy asphalt components on the surface of the aggregates.

The rate at which these, or possibly other reactions not specifically identified, occur appears to be extremely complicated and has been argued at length in the literature. For the engineer, these arguments may be superfluous because each reaction seems to lead to an undesirable change or embrittlement of the asphalt cement, which in turn has been associated with HMA having poor durability properties (*13*).

Many long term pavement performance studies involve periodic core sampling and testing to determine the changing asphalt cement properties such as penetration at 77°F (25°C) and viscosity at 140°F (60°C), and

pavement properties such as percent air voids in the pavement. Changes in such properties have been known to affect pavement performance with time, and have been found to follow a hyperbolic model (*15*, *16*, *17*). According to this theory, the changes in these physical properties follow a hyperbolic function with time and approach a definite limit as time increases. The following equation expresses the age hardening of asphalt in the field:

$$\Delta Y = \frac{T}{a + bT} \qquad \text{(Eq. 1)}$$

Or

$$\frac{T}{\Delta Y} = a + bT \qquad \text{(Eq. 2)}$$

where,

ΔY = change in test property (such as penetration and viscosity) with time T or the difference between the zero-life value and the value at any significant time;
T = time;
a = constant, the intercept of the Eq. 2 line on the ordinate;
b = slope of the Eq. 2 line; and
1/b = the ultimate change (limiting value of change) of the property at infinite time.

The extent of age hardening can be quantified in terms of penetration (percent retained penetration) or viscosity (aging index) as follows:

$$\%\ \text{Retained Penetration} = \frac{Penetration\ of\ aged\ asphalt}{Penetration\ of\ original\ asphalt} x100$$

$$\text{Aging Index} = \frac{Viscosity\ of\ aged\ asphalt}{Viscosity\ of\ original\ asphalt}$$

Both percent retained penetration and aging index have been used to evaluate relative aging of asphalt cements of different grades and/or from different sources.

The thin film oven (TFO) test and rolling thin film oven (RTFO) test generally simulate the asphalt cement hardening which occurs in the pugmill of a HMA batch facility. Because the asphalt cements from different sources age differently in service after pugmill mixing, the tests on TFO or RTFO residue are not completely reliable to predict the long-term asphalt aging or durability. There is an urgent need to develop an accelerated laboratory aging procedure for asphalt cements that can closely simulate the hardening that takes place in the pavement under certain climatic conditions (*18*). Field observations indicate that the asphalt cement does not age as rapidly in a drum HMA facility as it does in a batch facility because of the presence of water vapor in the drum, which reduces oxidation.

Penetration. The penetration test measures the consistency of paving asphalt cement which is expressed as the distance in tenths of a millimeter that a standard needle vertically penetrates into a sample of the asphalt cement under known conditions of loading, time, and temperature. Test procedures for measuring penetration at 77°F (25°C) and lower temperatures (usually 60°F or 16°C and 39.2°F or 4°C) are given in ASTM D5. It is an empirical test. In general, the penetration test involves higher stresses and shorter loading time than do most viscosity tests. Therefore, attention must be given to stress levels or shear rates, or both, to attain direct comparison of penetration and viscosity results. Penetration at 77°F (25°C) has been widely used in asphalt cement specifications because no simple method of viscosity determination at 77°F (25°C), or lower temperatures, is currently available. Even viscosity graded asphalt cement specifications require minimum penetration values at 77°F (25°C). As discussed earlier, penetration is also used to determine the temperature susceptibility of asphalt cements in terms of penetration index (PI) or pen–vis number (PVN).

Penetration at 77°F (25°C) generally gives the consistency of asphalt cement near average yearly service temperature, thus it has some influence on the overall performance of HMA pavement. A vast storehouse of data on penetration of aged asphalt cements in the pavements under service is available in the literature (*19*). It has been observed that aging of the HMA pavement results in progressively lower penetration with time as shown in Figure 2–25 (*20*). Hubbard and Gollomb (*21*) concluded the following from their study showing the effect of asphalt cement hardening on the development of cracks in HMA pavements:

1. When the penetration of the asphalt cement (77°F or 25°C) falls below 20, serious pavement cracking may occur.
2. Some cracking may occur when the penetration is between 20 and 30.
3. High resistance to cracking may occur when a mixture is well designed and properly compacted and the penetration of the asphalt cement is well above 30.
4. To ensure long life, one should use as soft an asphalt cement as possible without reducing stability below the minimum required to prevent displacement under traffic.

Most studies (*19*, *20*) generally support the above conclusions. Obviously, other factors such as asphalt cement film thickness, asphalt cement temperature susceptibility, asphalt cement ductility, air void content, age of the pavement, traffic, and climatic conditions also influence this general relationship between penetration and cracking.

Ductility. The ductility of a paving asphalt cement is measured by the distance to which it will elongate before breaking when two ends of a briquette specimen are pulled apart at a specified speed and temperature. ASTM D113 gives the test procedure to measure ductility at 77°F and lower temperatures.

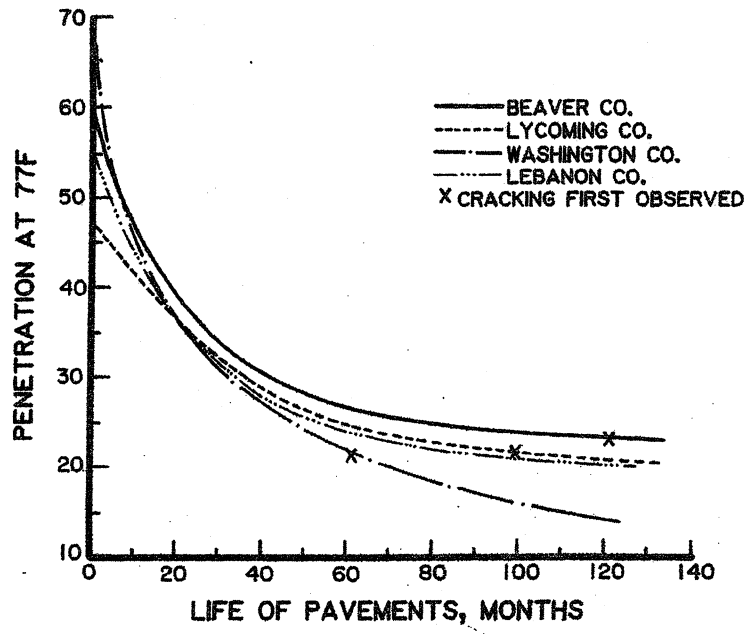


Figure 2-25. Penetration versus Time in Months (after Kandhal, *20*)

The significance of the ductility test as a means of asphalt cement quality control has been debated because of its empirical nature and poor reproducibility. Exactly which property is being measured is difficult to decide. Welborn et al (*22*) and Kandhal (*20*) reported good correlation between ductility and shear susceptibility at 45 and 60°F (7 and 16°C) for various asphalt cements, regardless of source. Barth (*1*) stated that the ductility is a valuable indicator of the physiochemical state of the asphalt colloid system.

Several studies have related the ductility of asphalt cement to pavement performance. Doyle (*23*) measured ductility at 55°F (13°C) and observed extensive pavement cracking when the ductility dropped below 5 cm. Halstead (*24*) demonstrated that the pavements containing asphalt cement with penetration in the range normally considered satisfactory (30 to 50), but with low ductilities are likely to show poorer service than pavements containing asphalt cements of the same

penetration, but with high ductilities. Reporting on the Zaca–Wigmore Project, Hveem et al (*25*) found increasing evidence that the ductility test on asphalt cements recovered from pavement during its service life is an important method for judging pavement performance. The ductility results for unsatisfactory asphalts on the Zaca–Wigmore Project were very low. Serafin (*26*) reviewed data on cores taken after 7 years service on the Michigan Bituminous Experimental Road and reported that the section containing the lowest ductility had the most pitting and cracking, whereas there did not appear to be any significant differences in penetration. Vallerga and Halstead (*27*) studied 53 highway pavements throughout the United States and reported that severe raveling occurred in cold climates when the ductility at 60°F (16°C) dropped to 3 cm or less. Kandhal (*20*) studied 10 test pavements in Pennsylvania and concluded that aging of the pavement results in progressively lower penetration and higher viscosity values. However, the accompanying decrease in low–temperature ductility was determined to be an important factor. After the penetration of asphalt cement drops below 30 because of hardening, the pavements containing asphalt cement with low ductilities showed poorer service than pavements containing asphalt cements of equal penetration but with high ductilities.

A subsequent study by Kandhal and Koehler (*18*) of six experimental test sections in Pennsylvania indicated that lower ductility values were associated with higher incidence of load–associated longitudinal cracking.

The preceding discussion indicates that the ductility test can discern the differences between different asphalt cements more effectively if it is run on highly stiff asphalt (either by lowering the test temperature below 77°F (25°C) and/or further aging of asphalt after TFO or RTFO testing). Present ASTM or AASHTO specifications require this test to be run at 77°F (25°C) on TFO residue. There is a need to develop a more fundamental test (such as shear susceptibility) for evaluating the quality of asphalt cement.

Viscosity. Viscosity at any given temperature and shear rate is essentially the ratio of shear stress to shear strain rate. At high temperatures such as 275°F (135°C), asphalt cements behave as simple Newtonian liquids; that is, the ratio of shear stress to shear strain rate is constant. At low temperatures, the ratio of shear stress to shear strain rate is not a constant, and the asphalt cements behave like non–Newtonian liquids. Unlike the empirical tests of penetration and ductility, viscosity is a fundamental consistency measurement in absolute units that is generally not affected by changes in test configurations or geometry of the sample (*28*). Standard ASTM test methods which use capillary viscometers are available to determine asphalt cement viscosity at 140°F or 60°C (ASTM D2171) and 275°F or 135°C (ASTM D2170) on a routine basis. Large vacuum capillary viscometers have been used to test asphalt cement viscosity at 77°F (*29*). The cone and plate viscometer is used in ASTM D3205 to measure viscosity in the range from 10^3 to 10^{10} poises and is therefore suitable for use at temperatures where viscosity is in the range indicated. The shear rate may vary between approximately 10^3 to 10^2 sec^{-1}, and the method is suitable for either Newtonian or non–Newtonian asphalt cements. This method is not used routinely at the present time.

Asphalt cement viscosity at 140°F (60°C) has some influence on the performance of HMA pavements during hot summer days when the pavement surface temperatures are near 140°F (60°C). A low viscosity at 140°F (60°C) can induce flushing and/or rutting if other factors are identical. This is especially true for mixes that have a relatively high asphalt content. Therefore, proper viscosity grade selection appropriate to prevailing climatic conditions must be made. For example, use of AC–10 in hot climates will most likely result in an increase in flushing and/or rutting of HMA pavements when compared to AC–20. On the other hand, an unusually high viscosity (or stiffness) at low service temperatures (such as -20°F or -29°C) in colder climates will result in non–load associated transverse shrinkage cracking.

It has been observed that aging of the HMA pavement results in progressively higher viscosity with time, as shown in Figure 2–24.

However, viscosity of the aged asphalt cement alone does not necessarily control the pavement performance. Aging indexes based on viscosity at 77°F (25°C) were found to be more meaningful to indicate comparative aging and pavement performance of test pavements in Pennsylvania. Raveling was observed on the test pavements when this aging index exceeded 12 (*11*).

Temperature Susceptibility. Asphalt cement is a thermoplastic material. Its consistency changes with temperature. Temperature susceptibility is the rate at which the consistency of an asphalt cement changes with a change in temperature and is a very important property of asphalt cement. Asphalt cements highly susceptible to temperature change are not desirable because (a) their viscosity at 275°F can be very low, resulting in occasional tender mix problems during compaction, and (b) their viscosity (stiffness) at the lowest service temperatures are usually very high, resulting in low temperature cracking. Three different approaches for determining temperature susceptibility of asphalt cement have been used. With the PG grading system these approaches are no longer needed since the properties of the asphalt binder are evaluated at low and high temperatures.

1. *Penetration Index (PI):*

Pfeiffer (*30*) expressed the temperature susceptibility quantitatively by a term designated as "penetration index" (PI).

The PI is determined from an asphalt cement's softening point (ring and ball test), its penetration at 77°F (25°C), and an assumption that the penetration of an asphalt cement at its softening point is 800. This is referred to as PI (pen/R&B). Several investigators have reported that the penetration of a substantial number of asphalt cements at their softening point can vary widely from 800, especially for waxy asphalts and asphalt cements having high softening points and high PI values. Therefore, it is considered prudent to measure penetration at some other temperature in addition to 77°F (25°C) rather than relying on this assumption.

Penetration can be measured at two temperatures to obtain this index. Logarithm of penetration is plotted against the test temperature in degrees C which normally gives a straight line. The slope A of this line is calculated as follows:

$$A = \frac{\log pen(T_1) - \log pen(T_2)}{T_1 - T_2}$$

The following empirical expression is then used to calculate the penetration index termed PI:

$$PI = \frac{20 - 500A}{1 + 50A}$$

Example 2-2: The penetration of an asphalt cement at 77°F (25°C) is 120. Its penetration at 40°F (4.4°C) is 10. What is the penetration index (PI)?

$$A = \frac{\log pen(T_1) - \log pen(T_2)}{T_1 - T_2} = \frac{\log 120 - \log 10}{25 - 4.4} = 0.052$$

$$PI = \frac{20 - 500A}{1 + 50A} = \frac{20 - 500(0.052)}{1 + 50(0.052)} = -1.67$$

Note that the temperatures used in the equation are in degrees centigrade.

The lower the PI value of an asphalt cement, the higher its temperature susceptibility. Most paving asphalt cements have a penetration index between +1 and –1. Asphalt cements with a PI below –2 are highly temperature susceptible, usually exhibit brittleness at low temperatures, and are very prone to transverse cracking in cold climates.

2. Pen–Vis Number (PVN):

McLeod proposed PVN to determine the temperature susceptibility of asphalt cements (*31*). This number is based on penetration at 77°F (25°C) and viscosity at either 275°F or 140°F (135°C or 60°C) which are usually specification requirements for viscosity graded asphalt cements,

thus the data is readily available to calculate the PVN. The following formula is used to calculate PVN:

$$PVN = \frac{L - X}{L - M}(-1.5)$$

where,

X = the logarithm of viscosity in centistokes measured at 275°F (135°C);

L = the logarithm of viscosity at 275°F (135°C) for a PVN of 0.0; and

M = the logarithm of viscosity at 275°F (135°C) for a PVN of –1.5.

The viscosity values of L and M can be read from the graph (Figure 2–26) developed by McLeod by plotting penetration at 77°F (25°C) against viscosity at 275°F (135°C) for "typical" asphalts. This graph contains two lines with the assigned PVN values of 0.0 and –1.5, which are used. However, the following equations (based on least squares line) can be used to calculate more accurate values of L and M:

The equation for the line representing a PVN of 0.0 is

$$\log V = 4.25800 - 0.79670 \log P$$

The equation for the line representing a PVN of – 1.5 is

$$\log V = 3.46289 - 0.61094 \log P$$

where,

V = the viscosity in centistokes at 275°F and P is the penetration at 77°F.

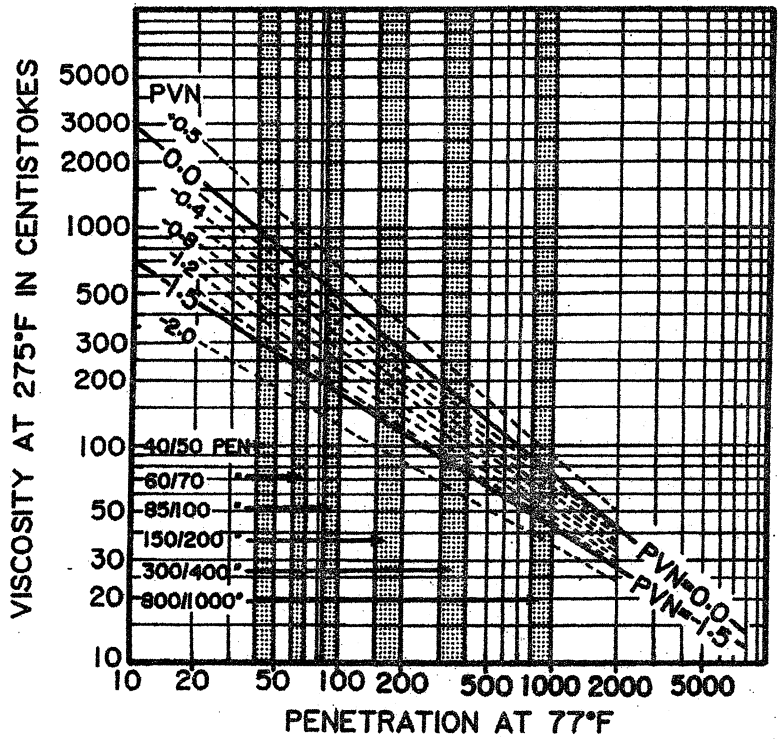


Figure 2-26. A Chart for the Determination of Approximate Values for PVN for Asphalt Cements (after McLeod, 31)

Example 2–3: An asphalt cement has a penetration of 100 at 77°F (25°C) and a viscosity of 400 centistokes at 275°F (135°C). What is its PVN? Use McLeod's chart (Figure 2–26).

$$PVN = \frac{L - X}{L - M}(-1.5)$$

X = log 400 = 2.602

L = log 450 = 2.653 (from McLeod's chart)

M = log 180 = 2.255 (from McLeod's chart)

$$PVN = \frac{2.653 - 2.602}{2.653 - 2.255}(-1.5) = -0.19$$

The lower the PVN value of an asphalt cement, the higher its temperature susceptibility. Most paving asphalt cements have a PVN between +0.5 to –2.0.

Although PVN can readily be determined from the specification tests, some asphalt technologists believe that the temperature susceptibility obtained in the higher temperature range (77–275°F) cannot be extrapolated to be applicable at temperatures below 77°F. One notable difference between PI and PVN is that the PI changes on aging (during mixing and subsequently in service), whereas the PVN remains substantially the same (32).

3. Viscosity–Temperature Susceptibility (VTS):

For determining VTS, a double logarithm of viscosity in centistokes is plotted against the logarithm of the absolute temperature in degrees K (empirical Walther's equation). Such plots generally result in straight lines, with the slope of the line being equal to VTS as follows:

$$VTS = \frac{\log\log vis(T_2) - \log\log vis(T_1)}{\log T_1 - \log T_2}$$

Larger VTS numbers indicate higher temperature susceptibility. Generally, the slopes of lines at temperatures lower than 140°F (60°C) tend to deviate from the slopes established between 140°F (60°C) and higher temperatures. This is because shear dependent viscosities are encountered below 140°F (60°C). The numerical differences between the VTS values of different asphalt cements are not large. The VTS value ranged from 3.36 to 3.98 for 52 asphalts sampled in the United States (33). Very little use of VTS is found in the literature.

Shear Susceptibillty. At high temperatures most asphalt cements exhibit purely viscous flow (Newtonian) in which rate of shear strain is proportional to stress. In other words, viscosity is independent of shear rate. However, at low temperatures and/or after aging, most asphalt cements exhibit non–Newtonian or viscoelastic flow. That is, their viscosity is dependent on the shear rate, the viscosity increases as the rate of shear increases. The rate of change of viscosity with rate of shear is referred to as shear susceptibility which is considered to be an intrinsic property of the asphalt cement. The shear susceptibility (or shear index) is the tangent of the angle of log shear rate (X – axis) versus log viscosity (Y – axis) plot determined by conducting the viscosity test at different shear rates. Shear susceptibility values of asphalt cements tend to increase at different rates on aging, depending on their chemical composition (*34*).

Shear susceptibility or viscosity of the aging asphalt alone does not necessarily indicate the pavement performance. The rate of gain in shear susceptibility relative to increases in viscosity at 77°F seems to be one of the major factors affecting pavement performance. Relatively lower gain in shear susceptibility with the corresponding increase in viscosity has been reported to be associated with better pavement performance (*17*).

Stiffness. Stiffness (or stiffness modulus) is the relationship between stress and strain as a function of time of loading and temperature; this relationship between stress, strain and time is also referred to as the rheological behavior of asphalt cements or mixtures. In many applications of HMA, its stiffness characteristics must be known not only to assess the behavior of the mix itself, but also to evaluate the performance of an engineering structure of which the mix is a part, such as a highway or airfield pavement (*13*). Ideally, for a highway pavement surface course, increased asphalt cement stiffness is desirable at high service temperatures (near 140°F or 60°C) to avoid rutting, and decreased asphalt cement stiffness is desirable at the low service temperature to resist low temperature shrinkage cracking. At high temperatures, increased resistance to rutting can be obtained by proper

selection of the aggregate component, mix design and proper selection of the asphalt cement stiffness. The asphalt cement stiffness is primarily responsible for cracking at low service temperatures.

Figure 2–27 shows a simplified diagram illustrating the time–of–loading dependence of the stiffness of HMA for a particular temperature. At very short loading times, it is noted that stiffness is essentially time independent; in this case, the stiffness approaches the elastic modulus. For an intermediate range on the time scale, the stiffness decreases with an increase in time of loading. At very long loading times, the stiffness may still decrease, but at a uniform rate, and the behavior may be considered to be purely viscous. The stiffness under these circumstances gives a measure of the flow characteristics of the mixture. In Figure 2–27, this viscous deformation has been characterized by viscous traction to indicate behavior under axial stress; as noted in the figure, this viscous traction is approximately three times the viscosity, which is a measure of flow resulting from shear stresses (*13*).

The response to a three–dimensional system of stress which occurs in a HMA pavement layer subjected to loading is rather complex. When the response also depends on the time or rate of loading and the temperature, the characterization is even more difficult (*13*).

In 1954, Van der Poel recommended to concentrate on a single stress and its resulting strain because for many purposes this is reasonably adequate (*35*). He suggested a single parameter termed stiffness (S) as follows:

$$S(t,T) = \frac{\sigma}{\varepsilon}$$

where,

S = stiffness, in psi or kg/cm^2;
σ = axial stress;
ε = axial strain;
t = time of loading; and
T = temperature.

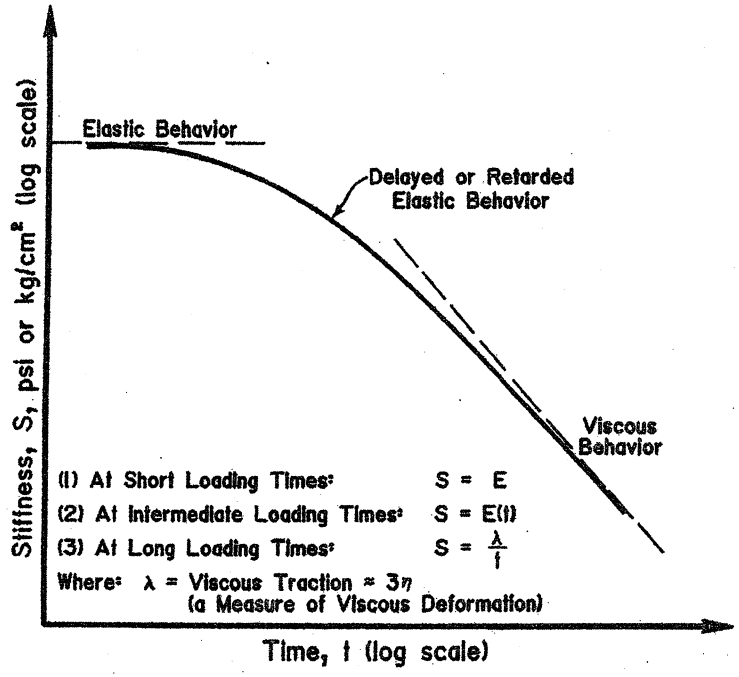

Elastic Behavior
Delayed or Retarded Elastic Behavior
Viscous Behavior
(1) At Short Loading Times: S = E
(2) At Intermediate Loading Times: S = E(t)
(3) At Long Loading Times: S = λ/t
Where: λ = Viscous Traction ≈ 3η
(a Measure of Viscous Deformation)
Stiffness, S, psi or kg/cm² (log scale)
Time, t (log scale)

Figure 2-27. Idealized Time–of–Loading Dependence of the Stiffness (Stress/Strain) Characteristics of an Asphalt Material Subjected to an Axial Tensile Stress (after Finn, *13*)

At very short times of loading and/or low temperatures, the behavior of HMA is almost elastic in the classical sense, and the stiffness, S, is analogous to an elastic modulus, E (Figure 2–27). At longer times of loading and higher temperatures, the stiffness is simply a relation between the applied stress and the resulting strain. Conversely, if the stiffness corresponding to a particular time and temperature and either the stress or the strain is known, an estimate can be made of the strain or the stress developed in the asphalt mixture (*13*).

Asphalt cement stiffness can be determined by the following two approaches:

1. Indirect Methods: These methods provide an estimate of stiffness without direct laboratory measurements. They use routine asphalt cement test data for estimating stiffness by use of nomographs.
2. Direct Methods: Asphalt cement stiffness is obtained by direct testing, by creep, relaxation, or constant–rate–of–strain testing in either tension or compression. Dynamic or flexural testing methods can also be used. The PG graded asphalts that will be discussed later use various stiffness tests to measure the binder stiffness at a number of temperatures. These tests are also direct methods of measuring stiffness.

A discussion of these two approaches follows.

1. *Indirect Methods:* Van der Poel conducted extensive creep and dynamic tests using well compacted (approximately 3 to 5 percent air voids) dense–graded HMA mixtures. He indicated that the stiffness of the mix is dependent on the stiffness of the asphalt cement which it contains and the volume concentration, Cv, of the aggregate (Figure 2–28).

Based on the test results, he developed a nomograph (Figure 2–29) which can be used to determine the stiffness of asphalt cement at any temperature and at any rate of loading if the softening point (R&B) and the penetration index PI (pen/R&B) of the asphalt cement are known. An "accuracy" on the order of a factor of 2 was reported and has been confirmed by some researchers. Once the asphalt cement stiffness is obtained from the nomograph, the stiffness of the HMA can be determined from Figure 2–29 using the volume concentration of the aggregate.

Heukelom and Klomp (*36*) studied Van der Poel's method in detail and modified the relationship between asphalt cement stiffness (S_{bit}) and mix stiffness (S_{mix}) as shown in Figure 2–30. Later Heukelom (*37*) suggested that the penetration be measured at three temperatures, plot the data on bitumen test data chart (BTDC), and determine the "corrected"

PI, which he called PI (pen/pen), and the "corrected" ring–and–ball softening point. The "corrected" softening point is obtained at the intersection of the extended penetration straight line and the 12,000 poise ordinate on the BTDC. These "corrected" softening points are then used in the Van der Poel nomograph. These corrections were deemed necessary because erroneous results are possible when the R&B softening point is used for waxy asphalt cements.

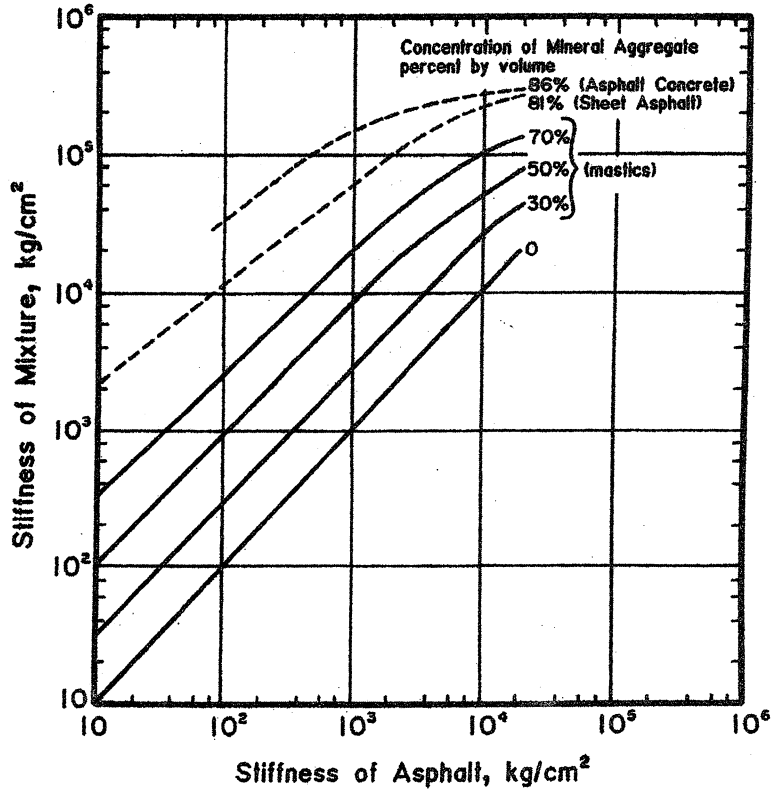


Figure 2–28. Relation between Stiffness of Asphalt Cement and Stiffness of Mixtures (after Van der Poel, *35*)

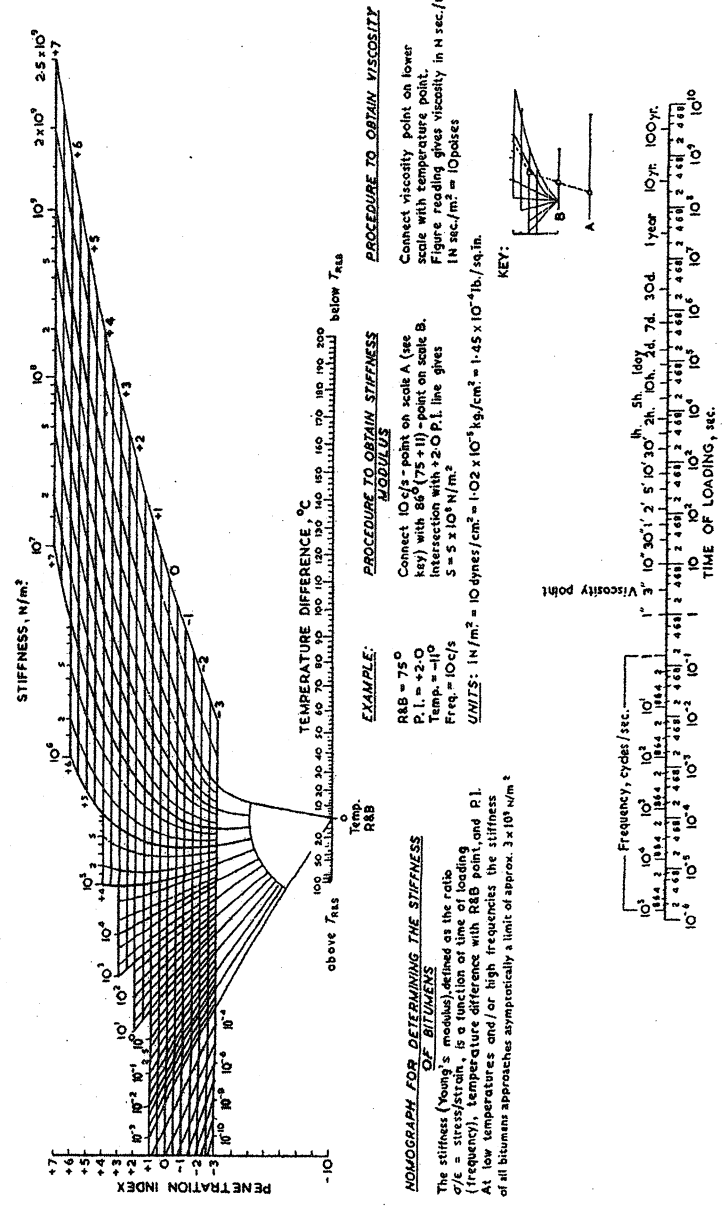


Figure 2–29. Nomograph for Determining the Stiffness of Bitumens (after Van der Poel, *35*)

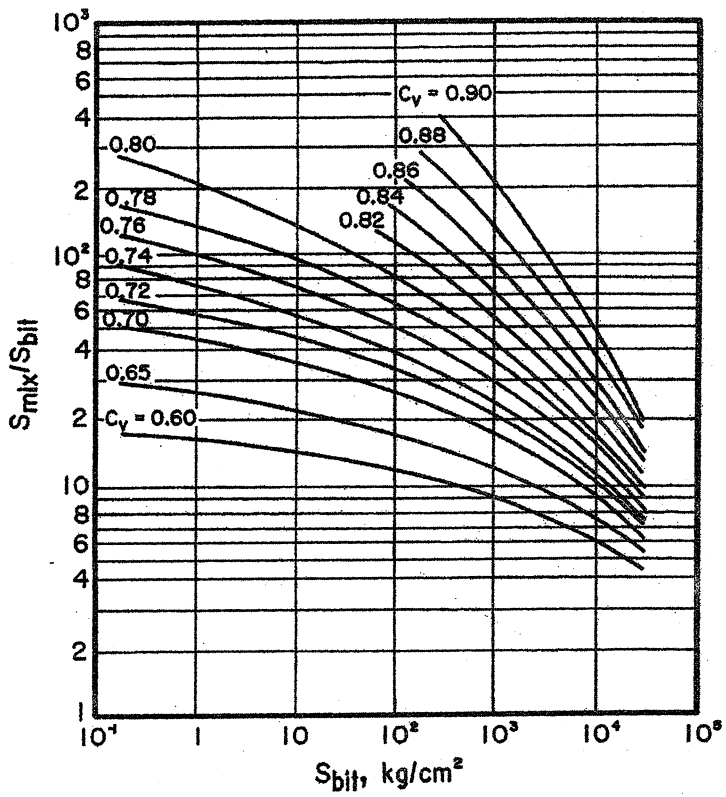


Figure 2–30. S_{mix} / S_{bit} as a Function of S_{bit} and Cv (after Heukelom and Klomp, *36*)

McLeod (*31*) further modified the Heukelom method for determining the stiffness of asphalt cement. The major difference is the use of pen–vis number (PVN) by McLeod in lieu of penetration index PI (pen/pen) used by Heukelom. Figures 2–31 and 2–32 are used to determine asphalt stiffness (*31*). McLeod's method requires measurement of asphalt cement penetration at 77°F (25°C) and viscosity at 275°F. Stiffness moduli values obtained by McLeod's method have been reported generally within a factor of 2 of the diametral creep stiffness moduli obtained by direct testing (*32*).

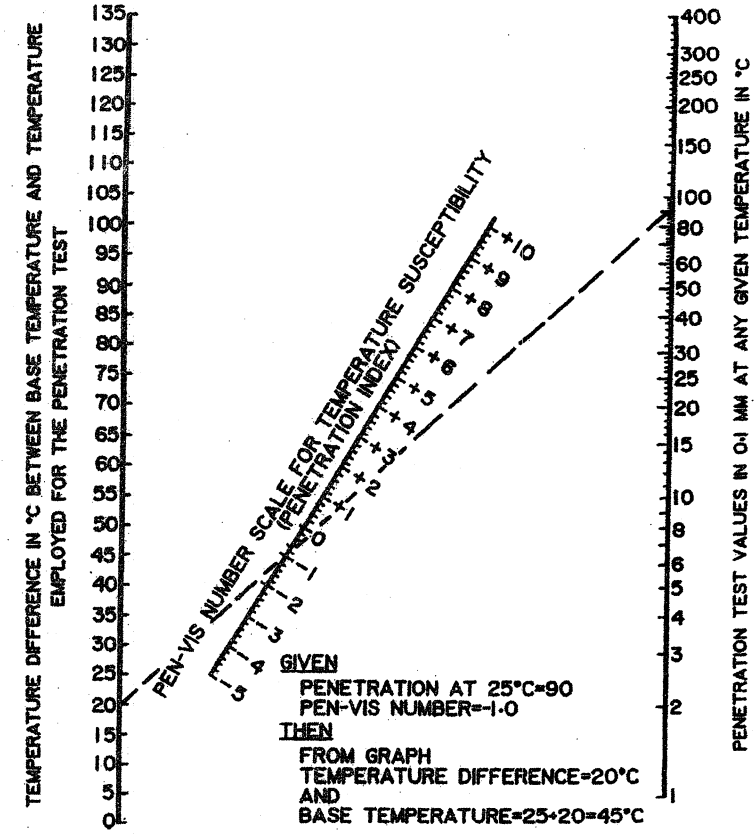

TEMPERATURE DIFFERENCE IN °C BETWEEN BASE TEMPERATURE AND TEMPERATURE EMPLOYED FOR THE PENETRATION TEST
PEN-VIS NUMBER SCALE FOR TEMPERATURE SUSCEPTIBILITY (PENETRATION INDEX)
PENETRATION TEST VALUES IN 0·1 MM AT ANY GIVEN TEMPERATURE IN °C
GIVEN
PENETRATION AT 25°C=90
PEN-VIS NUMBER=-1·0
THEN
FROM GRAPH
TEMPERATURE DIFFERENCE=20°C
AND
BASE TEMPERATURE=25+20=45°C

Figure 2–31. Relationship between Penetration, PVN, and Base Temperature for Asphalt Cements (after McLeod, *31*)

Although indirect methods are quick and easy to use, they have the following limitations (*38*):

a. The scale of nomographs requires that extreme care be used in interpolating the data and results. Appreciable errors can occur.
b. Determining stiffness of asphalt cement and mix by a series of transformations, starting from routine test data on the asphalt cement, appears to be somewhat "unscientific" to a number of engineers.

c. There is no estimate of possible errors in determining stiffness by indirect methods.

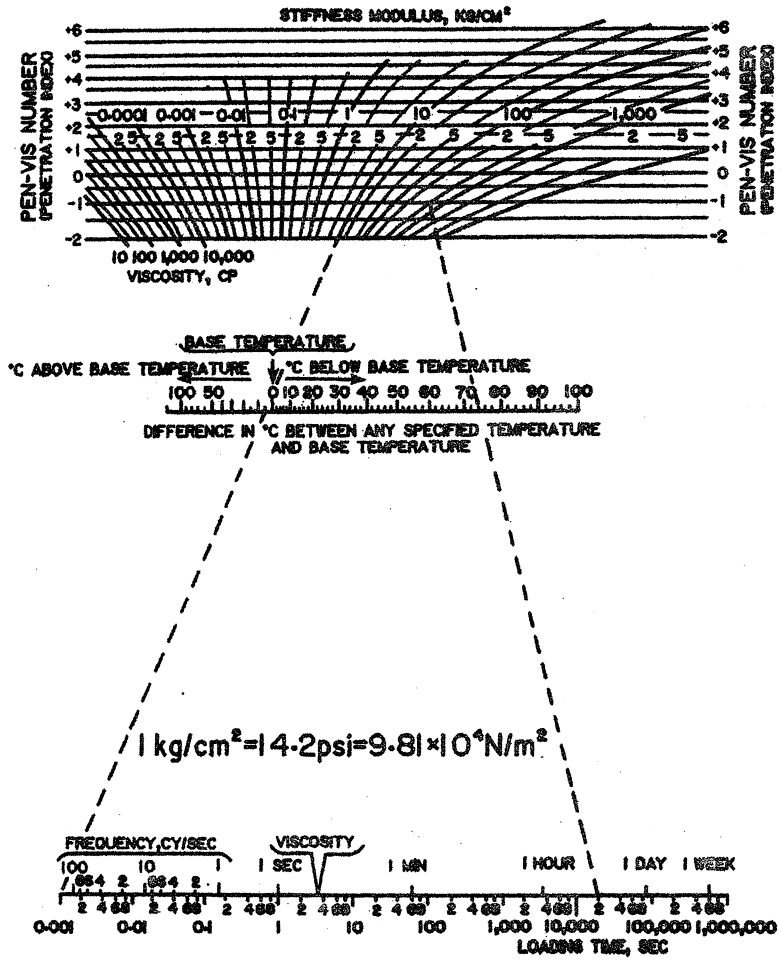


Figure 2–32. Nomograph for Determining Modulus of Stiffness of Asphalt Cement (after McLeod, *31*)

Despite the foregoing limitations, the indirect methods can be used for initial estimates of stiffness modulus if direct testing is not available.

2. *Direct Methods:* At high temperatures, viscosity can be used as a measure of asphalt cement stiffness. At 140°F (60°C) and higher, capillary viscometers are suitable because of the Newtonian behavior of asphalt cement in these temperature ranges. However, at low temperatures, a viscometer must be able to quantify viscosity as a function of both temperature and shear stress or rate. This complicated factor has resulted in the development of a large number of devices for measurement of shear–dependent viscosities (*28*).

In 1974, Schweyer (*39*) presented a pictorial overview of many experimental viscometers that have been used to study the viscoelastic behavior of asphalts. These included (a) rotational types, which utilize coaxial cylinders or a cone and plate; (b) rheometers, a specialized capillary type in which a piston is used to drive the asphalt through a capillary tube; and (c) other miscellaneous types. Schweyer and others have done considerable work on the use of the capillary rheometer (*40*) and the development of several generations of the constant–stress rheometer (*41*), which has been used to examine both asphalt cement stiffness and viscosity at low temperatures. Other devices that have been used include sliding plate rheometer (Figure 2-33) (*42, 43, 44*), rheometrics mechanical spectrometer (RMS) (*45*), and torsional rheometer.

The methods most widely used to determine the asphalt cement properties at low temperature are included as part of the PG grading system. These tests will be discussed in detail later in this chapter.

Pavement Performance Related to Asphalt Cement Rheological Properties

There are some specific types of HMA pavement distresses (affecting the pavement performance) which are related to the rheological properties of the asphalt cement. These distress types are discussed below.

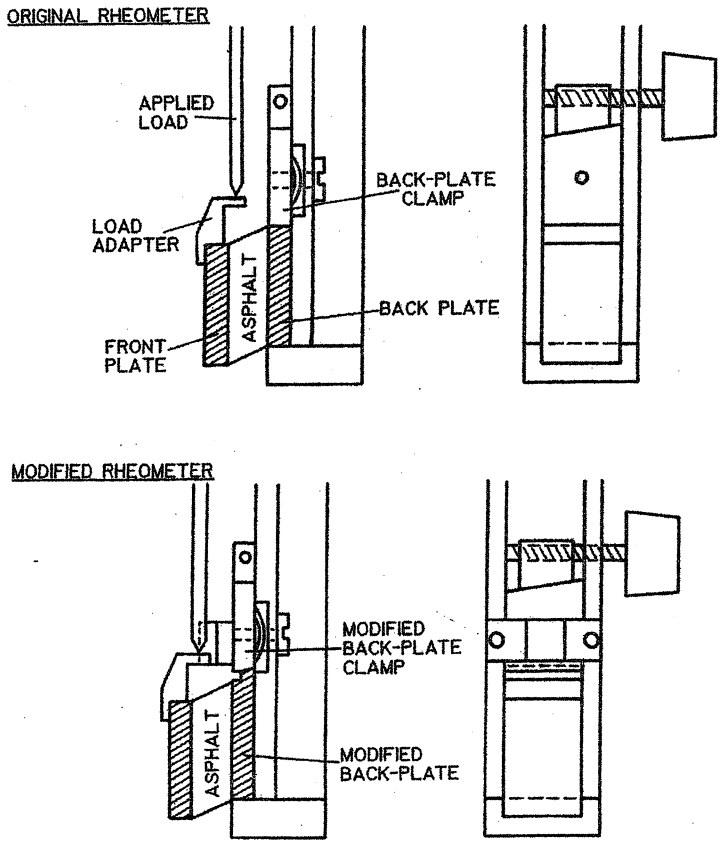


Figure 2–33. Original and Modified Rheometers (after Gaw, *44*)

Raveling. Raveling of a HMA pavement surface is usually caused by one or a combination of the following factors: (a) deficient asphalt content (lean mix), (b) insufficient amount of fine aggregate matrix to hold the coarse aggregate particles together, (c) lack of compaction (high air void content), (d) excessively aged (oxidized/brittle) or wrong grade of asphalt cement binder, and (e) lack of adhesion of the binger to the aggregate. Only the last factor is discussed here. It was mentioned earlier that age hardening of asphalt cement in pavement results in progressively

lower penetration or higher viscosity. This causes a progressive increase in the brittleness (and lack of binding characteristics) of asphalt cement and thus induces raveling. High air void contents in the HMA pavements, when constructed, accelerate the age hardening, and premature raveling results (*46*). The relationship between air void content and extent of raveling is shown in Figure 2–34. All other things being equal, asphalt cements from some sources age and harden faster than others and thus induce raveling earlier. Increased asphalt film thickness can significantly reduce the rate of aging and offset the effects of high air voids but other factors must be carefully considered when thinking about increasing the film thickness.

Cracking. Over the years, engineers have been able to categorize cracking into two broad groups: load associated and nonload associated, although most pavement cracks can be described according to their geometry such as longitudinal, transverse, polygon (also alligator and map) and block, or by the mechanism that causes the cracking such as slippage, shrinkage and reflection (*47*).

1. *Load–associated cracking*

The principal class of load–associated cracking has been described as fatigue cracking (alligator cracking), the phenomenon of fracture under repeated or fluctuating stress having a maximum value less than the tensile strength of the material. Some of the factors which influence the development of fatigue cracks are: (1) composition of structural section, (2) asphalt cement consistency, (3) asphalt content, (4) air voids and aggregate characteristics in asphalt mix, (5) in–place properties of untreated aggregates or cement–treated materials, and in–place properties of foundation soils, (6) temperature and (7) traffic (*47*). However, only asphalt cement properties will be discussed here.

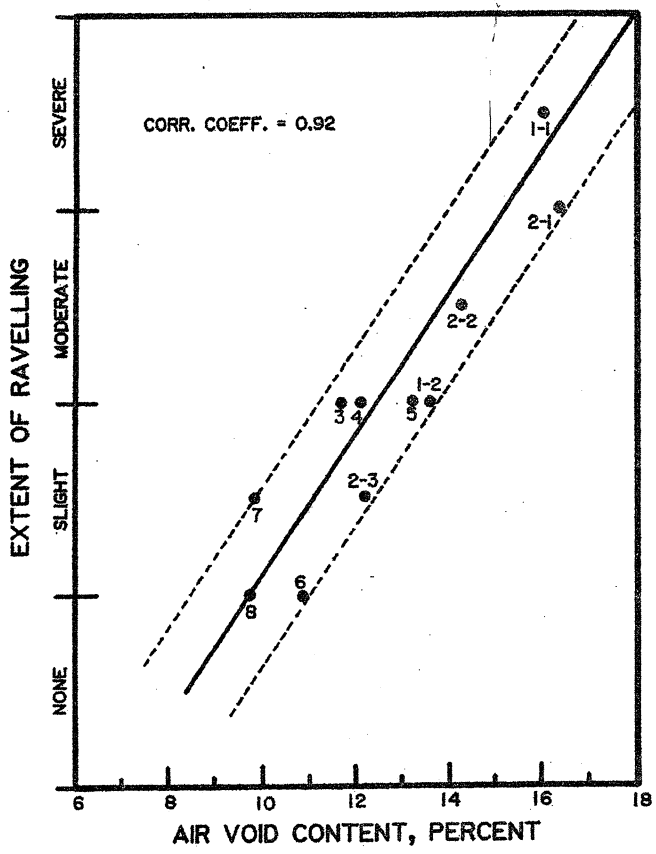


Figure 2–34. Air Void Content Versus Extent of Raveling (after Kandhal, *46*)

Numerous researchers have conducted fatigue tests on HMA mixes. However, laboratory test results have been influenced by the mode of testing (constant stress or constant strain) and the failure criteria. In general, constant stress tests will respond with an increasing fatigue life to any asphalt cement or mix property which increases the stiffness of HMA. For example, lower asphalt cement penetration or higher viscosity of asphalt cement generally result in a higher stiffness modulus and a

greater number of cycles to failure in the laboratory, all other things being equal. For constant strain tests, the effect of stiffness modulus is reversed; that is, higher penetration or lower viscosity of asphalt cement results in increased fatigue life. Based on the interpretation of numerous test results, the following hypotheses have been made relative to pavement thickness, stiffness modulus, and fatigue performance (*47*):

a. For thin asphalt pavement, mixes of low stiffness (low viscosity asphalt cements) should be used and the fatigue life should be evaluated in the constant strain mode of testing. Obviously, the stiffness should not be so low to result in rutting in the surface courses.
b. For thick pavements (125mm and greater), mixes of high stiffness (high viscosity asphalt cements) should be used and the fatigue life should be evaluated in the constant stress mode of testing. In recent years longitudinal top-down cracking in or near the wheel path has been observed in this type of pavement. These cracks initiate at the surface and propagate down to the bottom of the surface lift. Some studies indicate that these may be caused by the high shear stresses that are exerted by radial truck tires. Binders containing elastic modifiers have shown some benefit in mitigating this type of cracking.

This indicates that asphalt cements which become hard through overheating of the asphalt and/or aging will demonstrate poor fatigue life (develop fatigue cracking) in thin HMA surfacings.

2. *Nonload associated cracking*

Of all the many types of nonload associated cracking, low-temperature cracking is of most interest and concern to highway engineers (*48*). It manifests itself through transverse shrinkage cracking in the HMA pavement layer. Field observations and measurements have indicated the cracking starts at the surface and progresses down with

time. This happens because the low ambient temperatures chill the road surface first. HMA pavements subjected to high cooling rates and low temperatures develop tensile stresses due to shrinkage. If these stresses exceed the fracture strength of the HMA pavement layer, transverse cracking develops. HMA mixes which have high stiffness modulus at low temperatures are very prone to cracking. Mix stiffness at low temperatures is primarily dependent on the stiffness of the asphalt cement. Therefore, high asphalt cement stiffness at low temperatures is the predominant cause of this type of cracking. Other factors such as thickness of the HMA layer and subgrade soil type influence the frequency or extent of these cracks.

Prior to the PG grading system, laboratory research supported by field observations indicated that asphalt cement consistency, largely reported in terms of penetration at 77°F (25°C), and temperature susceptibility (PI or PVN) were the major asphalt cement properties influencing low temperature cracking. Increasing the asphalt cement penetration was suggested as the most direct designer–controlled variable for minimizing cracking. However, care must be exercised to avoid asphalt cements that are highly temperature susceptible (that is, asphalt cements that could become extremely stiff at low temperatures regardless of the initial penetration). Table 2 in AASHTO Specifications M226 for Viscosity Graded Asphalt Cements generally provides an asphalt cement with lower stiffness at lower temperatures than that provided in Table 1. However, Table 2 is not adequate for most states in the northern portions of the United States and Canada. More stringent specifications to control the temperature susceptibility of asphalt cements are needed to minimize or eliminate low temperature cracking.

Many researchers have recommended limiting stiffness values of asphalt cement (*43*, *49*, *50*) or HMA (*51*) for a particular temperature range based on field experiments. The Pennsylvania Department of Transportation modified Table 2 of AASHTO Specifications for Viscosity Graded Asphalt Cements in 1977, based on a limiting stiffness modulus of 275 kg/cm^2 (3900 psi) at the minimum pavement temperature of –10°F (–23°C) and 20,000 sec loading time (*50*). The limiting

stiffness modulus was established based on the performance of six test pavements. Figure 2–35 shows two test pavements in adjacent lanes. One developed extensive transverse cracks during the first winter and the other was free of cracks. Using the limiting stiffness of 275 kg/cm^2 and McLeod's nomograph method (*31*), minimum allowable PVNs were determined for various penetration values. Minimum kinematic viscosities were then determined from the corresponding penetration and PVN values. By specifying the minimum kinematic viscosity thus determined for each penetration value, it was ensured that the PVN was not lower than the permissible value. Some values are given below:

Penetration at 25°C	Minimum PVN Allowable	Specified Minimum Viscosity at 135°C centistokes
60	–0.80	390
65	–0.95	330
70	–1.10	290
75	–1.25	250

It can be seen that higher temperature susceptible asphalt cements can be used if their penetration values get higher in accordance with those listed above. At higher penetration values, the temperature viscosity lines are shifted relatively lower at 77°F (25°C), thus the asphalt cement does not exceed the limiting stiffness modulus at –10°F (–23°C) in spite of steeper slopes (higher temperature susceptibility) of these lines.

The St. Anne Test Road in Canada has been the most comprehensive full scale project reported in the literature (*52*). Many researchers have based the critical asphalt cement stiffness at low temperatures on data from this project (*43*). The St. Anne test pavements cracked at an asphalt binder stiffness of 1x10^9 N/m^2 (145,000 psi) at 0.5–h loading time (that is, equivalent to approximately 6x10^8 N/m^2 at 2.0–h loading time; 5x10^8 N/m^2 at 10,000–s loading time). Readshaw (*53*) based the British Columbia asphalt cement specifications on a critical asphalt cement

stiffness of $2x10^8$ N/m^2 at 2.0 h loading time, a value arrived at largely from the St. Anne Test Road data. Fromm and Phang (*49*) suggested $1.4x10^8$ N/m^2 (that is, 20,000 psi) at 10,000–s loading time as the critical asphalt cement stiffness.

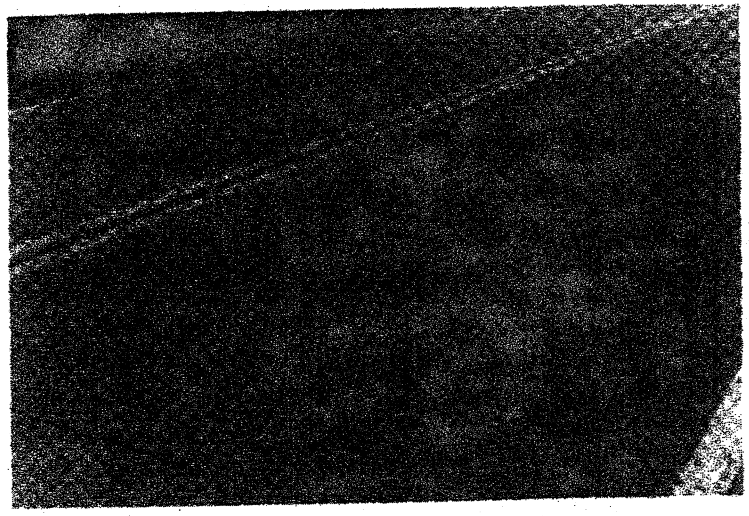


Figure 2–35. Test Pavements in Adjacent Lanes—Foreground Lane Shows Transverse Cracks, No Cracks in the Background Lane (after Kandhal, *50*)

Gaw (*43*) has recommended that the low–temperature asphalt cement specifications be based on a specific nomographic cracking temperature selected from Figure 2–36, at a performance level appropriate to the anticipated minimum ambient temperature or pavement surface temperature for the region. The asphalt cement specification limits are represented by a single line drawn on a logarithm plot of asphalt cement penetration at 25°C (77°F) and 5°C (41°F) similar to Figure 2–36. Candidate asphalt cements for which the penetration at 25 and 5°C appear on, or to the right of, the specification line are considered acceptable. Both unaged and aged asphalt cement penetrations yield similar nomographic cracking temperatures, and either can be used to

establish the specification limits. This approach requires the penetration measurement at one additional temperature (that is, at 5°C).

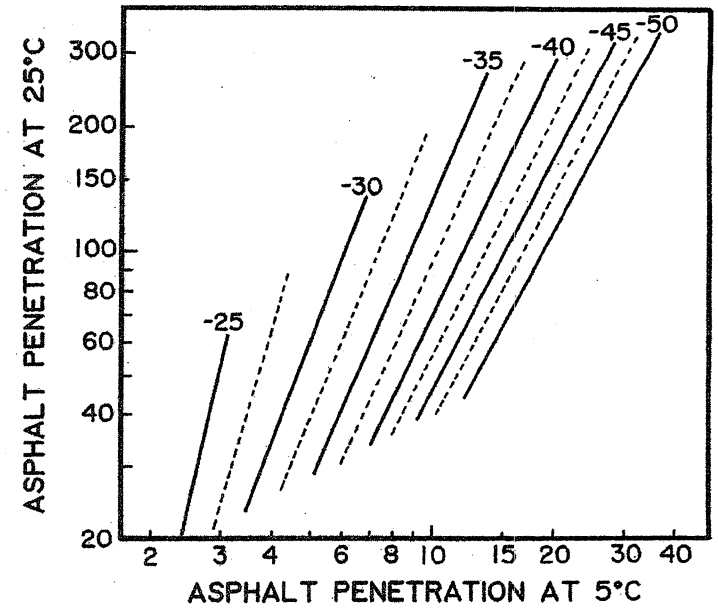


Figure 2–36. Nomograph for Predicting Cracking Temperatures in °C from asphalt Penetration (after Gaw, *43*)

Again the latest approach recommended is to use PG graded asphalts which measure properties at low temperatures instead of extrapolating the properties from tests at higher temperatures as discussed above. The PG grading system, discussed later, is the best approach to control the mix performance at low and high temperatures but especially at low temperatures.

Rutting. Rutting is the permanent deformation of the HMA and/or the underlying base or subbase caused by repeated traffic loads. This can occur either through plastic flow in the HMA layer or shear in one of the underlying layers. Rutting can also be caused by consolidation of the pavement layers by traffic after construction has been completed.

When compaction is poor (10 to 12 percent or more air voids for a mix designed at 3 to 5 percent), the channelized traffic provides a repeated kneading action in the wheel track areas and completes the consolidation to the designed air voids level (usually 3 to 5 percent).

If the asphalt mixture is not properly manufactured and compacted, rutting can also result from lateral plastic flow (permanent deformation) of the HMA layer caused by the high shear force of radial truck tires, extended high temperature, and slow moving traffic (e.g. intersections). Use of excessive asphalt cement could be a cause for this phenomenon. Too much asphalt cement in the mix causes the loss of internal friction between aggregate particles and results in the loads being carried by the asphalt cement rather than the aggregate structure. Rutting can also be the result of using rounded aggregates such as naturally occurring sands and gravels. During construction it is important that the mixture be controlled to ensure that excessive rounded material is not used in the mixture.

The high temperature stiffness of the asphalt cement is very important to minimize plastic flow in the HMA. Increased resistance to rutting can be obtained by using an asphalt cement which is stiffer at high temperature (e.g. higher PG grading, higher viscosity, or lower penetration). However, stiffer asphalt cements are more prone to cracking during winter in cold regions, especially in the surface courses. Therefore the low temperature grade must be controlled when using the PG grading system to ensure that satisfactory low temperature properties are obtained. Certain mineral fillers (or baghouse fines) can also increase the stiffness of asphalt mastic (*54*) and thus make the mix more resistant to rutting. However, these same fillers will tend to close the voids in the mixture if not properly controlled also resulting in potential rutting problems.

Stripping. Stripping (more appropriately called moisture–induced damage) can be defined as the weakening or eventual loss of the adhesive bond usually in the presence of moisture between the aggregate surface and the asphalt cement in an HMA mixture. Strength of the mixture is

derived from the cohesion of the binder and grain interlock and internal friction of the aggregate. The cohesion is only fully available if a good bond exists between the binder and the aggregate (*55*). If the bond is poor, failure occurs at the binder–aggregate interface and may result in premature failure of the mixture and the HMA pavement.

There may be as many as five different mechanisms by which stripping of asphalt cement from an aggregate surface may occur. Those five mechanisms include detachment, displacement, spontaneous emulsification, pore pressure, and hydraulic scouring (*56*). The stripping phenomenon is discussed in detail later in the book.

Stripping is a complex problem depending on many variables, including the type and use of mix, asphalt cement characteristics, aggregate characteristics, environment, traffic, construction practice, and the use of antistrip additives; however, the presence of moisture under traffic loading is the common factor to all stripping (*56*, *57*, *58*).

The chemistry of both the asphalt and aggregate at the asphalt–aggregate interface plays an important role in the stripping phenomenon. The effect of asphalt cement characteristics especially the physical properties, although secondary in role, is discussed here. The most frequently referenced relationship between the characteristics of the asphalt cement in a paving mixture and the tendency of the mix to strip relates stripping resistance to the viscosity of the asphalt binder in service. Binders of high viscosity have generally been observed to resist displacement by water much better than those of low viscosity. Low viscosity, however, is desirable during mixing operations because a low viscosity fluid has more wetting power than one of high viscosity. The viscosity is a function of the grade of the asphalt and the temperature to which it is heated. Moreover, high viscosity asphalt cements cannot be used in many instances because of other considerations such as low–temperature cracking in cold regions and potential reduction in fatigue life of the surface courses. There is a need to understand the fundamentals of aggregate–asphalt adhesion so that the stripping problem can be minimized by other means rather than increasing the

asphalt cement viscosity. This is not effective in all cases and may result in other performance problems.

SUPERPAVE ASPHALT BINDER TESTS AND SPECIFICATIONS

The asphalt cement physical tests and specifications for penetration and viscosity graded asphalt have many limitations as noted below.

1. Tests such as penetration and ductility are empirical and are not directly related to HMA pavement performance. Any relationship between the empirical tests and performance has to be gained by experience as discussed previously.
2. Tests are conducted at one standard temperature such as penetration at 77°F (25°C) and viscosity at 140°F (60°C) in spite of different climatic conditions at project sites or geographical regions.
3. The entire range of pavement temperatures typically prevalent at a project site is not considered. For example, with penetration or viscosity grading tests there is no test method or specification requirements for asphalt cement stiffness at low temperatures to control thermal cracking.
4. Penetration and viscosity grading test methods and specifications consider short term aging of asphalt cements during HMA production and construction by conducting thin film oven (TFO) or rolling thin film oven (RTFO) tests. However, long term aging of asphalt cements in service is not considered. As discussed previously, asphalt cements continue to stiffen during service thereby affecting the HMA performance in terms of fatigue cracking and low temperature cracking.
5. Asphalt cements can have different temperature and performance characteristics within the same grading system whether it is a penetration grading or a viscosity grading system.

6. For penetration and viscosity grading, tests and specifications have been developed for asphalt cements based on experience. They are not suitable or applicable to modified asphalt binders which are gaining in popularity. See asphalt additives and modifiers in Chapter 7.

Recognizing the preceding limitations of the penetration and viscosity grading systems, a 5–year Strategic Highway Research Program (SHRP) was launched in 1987, which included a $50–million research effort to develop performance– based tests and specifications for asphalt binders and HMA mixtures. Based on work under this program the Superpave (Superior Performing Asphalt Pavements) mix design system and the PG (performance graded) asphalt binder grading system was developed in the mid 1990s. The PG grading system has been almost entirely adopted in the United States and in many other countries as well. The PG grading system binder tests and specifications (*59*, *60*) have the following features.

1. Tests and specifications are intended for asphalt "binders" which include both modified and unmodified asphalt cements.
2. The specified test criteria for the asphalt binder remains constant, however, the temperature at which the criteria must be met changes in consideration of the binder grade selected. The binder grade is typically selected for given climatic conditions and traffic levels.
3. The physical properties measured by Superpave binder tests are directly related to field performance by engineering principles.
4. The Superpave binder specification requires the asphalt binder to be tested after simulating its three critical stages: (a) the first stage is represented by original asphalt binder which has to be transported, stored, and handled prior to mixing with aggregate, (b) the second stage is represented by the aged asphalt binder after HMA production and construction (short–term aging), and

(c) the third stage is represented by the asphalt binder which undergoes further aging during a long period of time in service.

5. The entire range of pavement temperatures experienced at the project site is considered.
6. Tests and specifications are designed to provide an asphalt binder grade that optimizes performance related to three specific types of HMA pavement distresses: rutting, fatigue cracking, and thermal cracking. Rutting typically occurs at high temperatures, fatigue cracking at intermediate temperatures, and thermal cracking at low temperatures.
7. The Superpave asphalt binder test procedures and specifications were developed in SI units which will be used in this section without English units.

Physical Tests for PG Graded Asphalt Binders

Table 2–2 gives the list of testing equipment to conduct various physical tests for PG grading, the related purpose for testing, and the related performance parameter being partly influenced by the asphalt binder. It must be recognized that the HMA mixture properties and the pavement structure significantly affect the HMA performance in addition to asphalt binder properties.

Rolling Thin Film Oven (RTFO). The rolling thin film oven (RTFO) test, AASHTO T240 and ASTM D2872, was described earlier in this chapter. Like the thin film oven (TFO) test, the RTFO simulates the asphalt binder aging during the manufacture and construction of HMA pavements. The RTFO was selected because (a) it continually exposes fresh binder to heat and air flow during rolling, (b) modifiers, if used, usually remain dispersed in the asphalt binder due to rolling action, (c) unlike TFO test in which the binder does not move, it does not allow any surface skin to be formed, which inhibits aging, and (d) it takes only 85 minutes to perform rather than five hours required for the TFO test.

The RTFO test provides an aged asphalt binder for further testing by the dynamic shear rheometer (DSR), and allows the determination of the mass of volatiles lost from the binder during the test. The amount of volatiles lost is related to the amount of aging that may occur during HMA production and construction. Some asphalt binders gain weight, rather than lose weight, during the RTFO aging because of the oxidative products formed during the test.

Table 2-2. Superpave Asphalt Binder Testing Equipment and Purpose

Equipment	Purpose	Performance Parameter
Rolling Thin Film Oven (RTFO)	Simulate binder aging (hardening) during HMA production and construction	Resistance to aging (durability) during construction
Pressure Aging Vessel (PAV)	Simulate binder aging (hardening) during HMA service life	Resistance to aging (durability) during service life
Rotational Viscometer (RV)	Measure binder properties at high construction temperatures	Handling and pumping
Dynamic Shear Rheometer (DSR)	Measure binder properties at high and intermediate service temperatures	Resistance to permanent deformation (rutting) and fatigue cracking
Bending Beam Rheometer (BBR)	Measure binder properties at low service temperatures	Resistance to thermal cracking
Direct Tension Tester (DTT)	Measure binder properties at low service temperatures	Resistance to thermal cracking

Pressure Aging Vessel (PAV). Pressure aging has been used in asphalt research and for aging rubber products for many years. A method

involving the pressure aging was developed by Dr. D. Y. Lee at Iowa State University for long term aging of asphalt cements, and is currently a standard method for aging rubber products (ASTM D454 and ASTM D572 in Reference 61). Pressure aging of asphalt binders has the following advantages: (a) limited loss of volatiles, (b) the oxidative process is accelerated without resorting to high temperatures, (c) an adequate amount of asphalt binder can be aged at one time for further testing, and (d) the test is practical for routine laboratory testing.

The pressure aging vessel (PAV) was developed in SHRP to simulate the asphalt binder aging that occurs during 5–10 years of in–service HMA pavements (62). Since the asphalt binder in the HMA pavement has undergone short–term aging during production and construction, the PAV is used to age RTFO residue. Figure 2–37 shows a schematic of the PAV and related accessories.

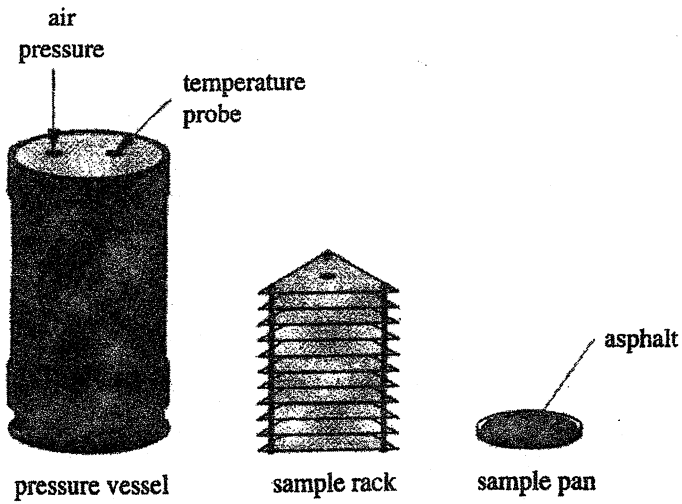


Figure 2–37. Pressure Aging Vessel (59)

The RTFO residue is exposed to high air pressure and temperature for 20 hours in the PAV to simulate the effect of long term aging. A cylinder of clean, dry compressed air with a pressure regulator, release

valve, and a slow–release bleed valve is used to supply and regulate air pressure. The PAV composed of stainless steel must be able to operate under the pressure (2070 kPa) and temperature conditions (90, 100, or 110 C) of the test. The vessel accommodates a sample rack on which ten sample pans can be placed for aging. The PAV is placed in an oven for 20 hours to maintain the desired temperature inside the vessel. PAV equipment which incorporates the pressure aging chamber and the oven built as one unit, is also available.

RTFO–aged asphalt binder is heated until fluid, stirred and placed in the PAV pan which can accommodate a 50–gram sample. The sample rack containing sample pans is placed in an unpressurized PAV preheated to the desired test temperature depending on the design climate. The lid of the PAV is secured quickly to minimize heat loss. When the PAV reaches within 2°C of the desired temperature, a pressure of 2070 kPa is applied using the valve on the air cylinder. After 20 hours, the air pressure is released slowly (over a period of 8–10 minutes) using the bleed valve. A quick pressure release may cause foaming in the binder samples. The sample pans are then placed in an oven maintained at 163°C for 30 minutes to remove entrapped air from the samples. The samples are then stored for further testing (such as dynamic shear rheometer, bending beam rheometer, and direct tension tester).

Rotational Viscometer (RV). The rotational viscometer (RV) has been adopted in Superpave for determining the viscosity of asphalt binder at high construction temperatures (above 100°C) to ensure that the binder is sufficiently fluid for pumping and mixing. Most asphalt binders behave as Newtonian fluids (stress response not dependent on shear rate) and have a totally viscous response at such high temperatures. Therefore, a viscosity measurement is sufficient to represent workability of the binder (63). The RV is more suited for testing many modified asphalt binders (such as those containing crumb rubber modifier) compared to capillary viscometer because the latter can get clogged up partially inhibiting smooth flow. The Superpave binder specification limits the viscosity to 3 Pa.s at 135°C.

This method of measuring viscosity is given in detail in ASTM Method D4402 or AASHTO TP48, "Viscosity Determination of Unfilled Asphalts Using the Brookfield Thermosel Apparatus." Figure 2–38 shows the principle of rotational or Brookfield viscometer. Rotational viscosity is determined by measuring the torque required to maintain a constant rotational speed (20 RPM) of a cylindrical spindle while submerged in an asphalt binder at a constant temperature. The measured torque is directly related to the viscosity of the binder sample which is automatically determined and displayed by the viscometer.

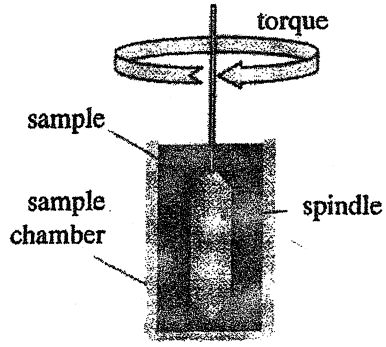


Figure 2–38. Principle of Rotational Viscometer (*59*)

The test equipment to measure rotational viscosity consists of two items as shown in Figure 2–39: Brookfield viscometer and Thermosel™ system. The Brookfield viscometer is comprised of a motor, spindle, control keys, and digital readout. Many sizes of the spindle are available for the Brookfield viscometer to suit the viscosity range of the binder being tested. Most asphalt binders can be tested with only two spindles: Nos. 21 and 27, the latter is used most frequently. The Thermosel™ system consists of a stainless steel sample chamber, a thermo container equipped with electric heating elements, and a temperature controller to set the test temperatures.

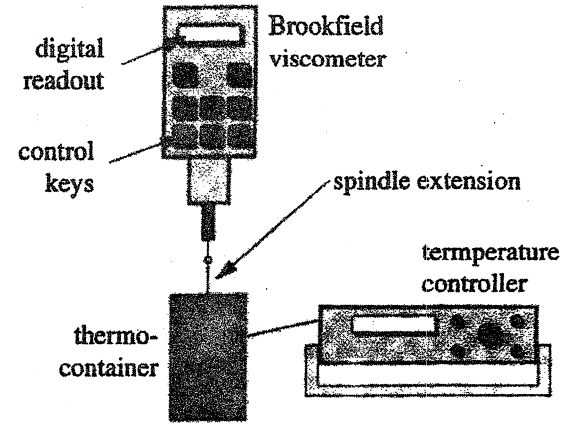


Figure 2–39. Schematic of Brookfield Viscometer (*59*)

Approximately 30 g of asphalt binder is heated in an oven so that it is sufficiently fluid to pour. The binder is weighed into the sample chamber. The amount of binder used varies with the size of the spindle. Less binder will be required with a larger spindle. Typically, no more than 11 grams are used. The sample chamber containing the binder sample is then placed in the thermo container and is ready for testing when the desired temperature (135°C for specification) stabilizes. The spindle is lowered into the chamber containing hot binder sample, and is coupled with the viscometer. After the sample temperature stabilizes again, the viscometer motor is turned–on at 20 RPM. The viscometer reading is displayed digitally along with spindle number, RPM, and temperature. The digital display of the rotational viscosity is in units of centipoise (cP) whereas the Superpave binder specification uses Pa.S. The following factor is used to convert from cP to Pa.s.

$$1000\ cP = 1 Pa.s$$

Therefore, the Brookfield viscosity output in cP is multiplied by 0.001 to obtain viscosity in Pa.s.

Most agencies use equiviscous temperatures for mixing and compaction during HMA mix design (see Chapter 4). This can be

accomplished by measuring rotational or Brookfield viscosity at one more temperature other than 135°C, and plotting the temperature–viscosity curve.

Dynamic Shear Rheometer (DSR). The dynamic shear rheometer (DSR) is used to characterize the viscous and elastic behavior of asphalt binders at high and intermediate service temperatures. The DSR which is also generically known as dynamic shear rheometer or oscillatory shear rheometer, has been used in the plastics industry for many years. The DSR measures the complex shear modulus G* (G star) and phase angle δ (delta) of asphalt binders at the desired temperature and frequency of loading. Complex modulus G* can be considered as the total resistance of the binder to deformation when repeatedly sheared. As shown in Figure 2–40, complex modulus G* consists of two components: (a) storage modulus G' (G prime) or the elastic (recoverable) part, and (b) loss modulus G" (G double prime) or the viscous (non–recoverable) part (*59*).

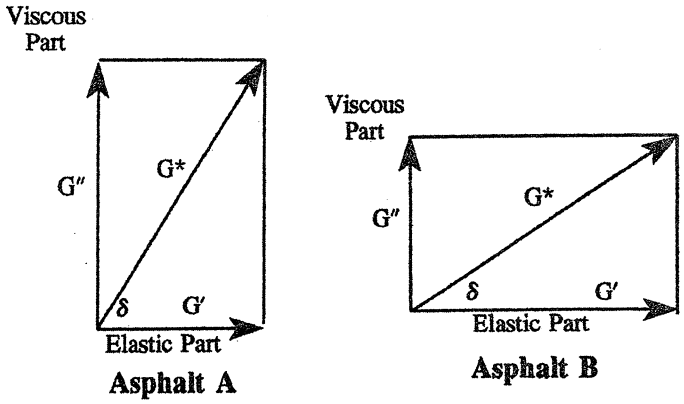


Figure 2–40. Components of Complex Modulus G* (*59*)

Both temperature and frequency of loading significantly affect the values of G* and δ for asphalt binders. Asphalt binders behave like elastic solids as shown by the horizontal arrow in Figure 2–41 at very

low temperatures. On the other hand, asphalt binders behave like viscous fluids at high temperatures (well above pavement temperatures) as shown by the vertical arrow in Figure 2–41. However, at usual pavement service temperatures most asphalt binders behave like a viscoelastic material (represented by arrows A and B in Figure 2–41), that is, they simultaneously act like elastic solids and viscous liquids. When viscoelastic materials are loaded, part of the deformation is elastic (recoverable) and part is viscous (non–recoverable). The DSR gives a complete picture of the behavior of asphalt binders at pavement service temperatures by measuring both G* and δ.

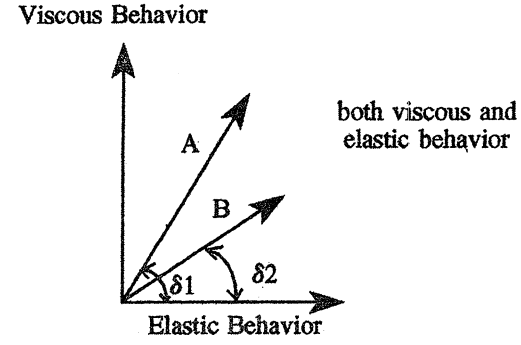


Figure 2–41. Viscous and Elastic Behavior of Asphalt Binders (*59*)

Figure 2–40 shows two asphalt binders with the same complex modulus G* (indicated by length of the diagonal) but with different phase angles δ. However, Asphalt B has a larger elastic component compared to Asphalt A. When both asphalt binders are loaded, Asphalt B will display more elastic (recoverable) deformation and less viscous or non–recoverable (permanent) deformation compared to Asphalt A. This example demonstrates that complex modulus (G*) alone is not sufficient to characterize asphalt binders, phase angle (δ) is also needed.

The elastic component or storage modulus is related to the amount of energy stored in the sample during each testing cycle. The viscous component or loss modulus is related to the energy lost during each

testing cycle through permanent flow or deformation (*59*, *60*). It is important to note that a real viscoelastic material does have a significant amount of delayed elastic response which is recovered completely when the sample is no longer loaded. Both the storage and loss moduli contain a portion of this delayed elastic response. In this sense, the storage and loss moduli cannot be considered purely elastic or viscous respectively (*64*).

The principle of operation of DSR is simple. The test procedure is given in AASHTO TP5. As shown in Figure 2–42, the asphalt binder sample is sandwiched between a fixed plate and an oscillating plate. When torque is applied to the oscillating plate, it starts from point A and moves to point B. From point B, the plate moves back and goes to point C passing the point A. From point C it returns back to point A. This movement comprises one cycle of oscillation. If two oscillation cycles occur in one second, the frequency of oscillation would be two cycles per second or two hertz (Hz). The frequency of oscillation can also be expressed as the circumferential distance (radians) traversed by the oscillating plate in one second. All Superpave DSR tests are conducted at a frequency of 10 radians per second which is equivalent to about 1.59 Hz.

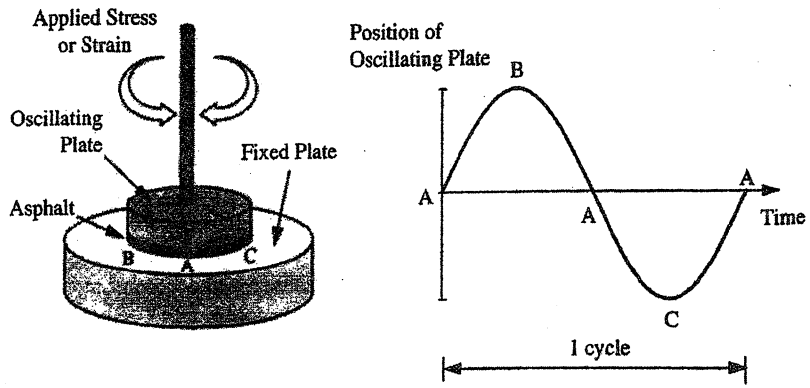


Figure 2–42. Basics of Dynamic Shear Rheometer (*59*)

Two types of oscillatory shear rheometers are available: constant stress and constant strain. Constant stress rheometers use a fixed torque to oscillate the top plate or spindle and, therefore, the strain will vary. Constant strain rheometers work by moving the plate a fixed distance from point A to point B and measuring the necessary torque to cause this movement. Superpave binder tests are conducted in the constant stress mode.

The thickness of asphalt binder sample sandwiched between the fixed plate and the oscillating spindle depends on the test temperature. High test temperatures (46°C or greater) require a small gap of 1000 microns (1 mm). Lower test temperatures in the 4° to 40°C range require a larger gap of 2000 microns (2 mm). Also, two spindle diameters are used: large spindle (25 mm) for high test temperatures and small spindle (8 mm) for low test temperatures (*59*).

After the asphalt binder sample is sandwiched between the fixed plate and the spindle the desired test temperature has to be achieved within ±0.1°C using a water or air bath. When the spindle is oscillated back and forth with constant stress, the resulting strain is monitored. The relationship between the applied stress and the resulting strain (Figure 2–43) is used to compute G*. G* is the ratio of maximum shear stress (τ_{max}) to maximum shear strain (γ_{max}) or $\tau_{max} \div \gamma_{max}$. The time lag between the applied stress and the resulting strain is the phase angle δ. For a perfectly elastic material, there is an instant response and, therefore, the time lag between the applied stress and the resulting strain or phase angle δ is zero. When a viscous liquid such as hot asphalt binder is tested, the time lag is large and the phase angle δ approaches 90 degrees. However, asphalt binders display a stress–strain response between these two extremes at pavement service temperatures as shown in Figure 2–44. The resulting phase angle is between 0 to 90 degrees (*59*).

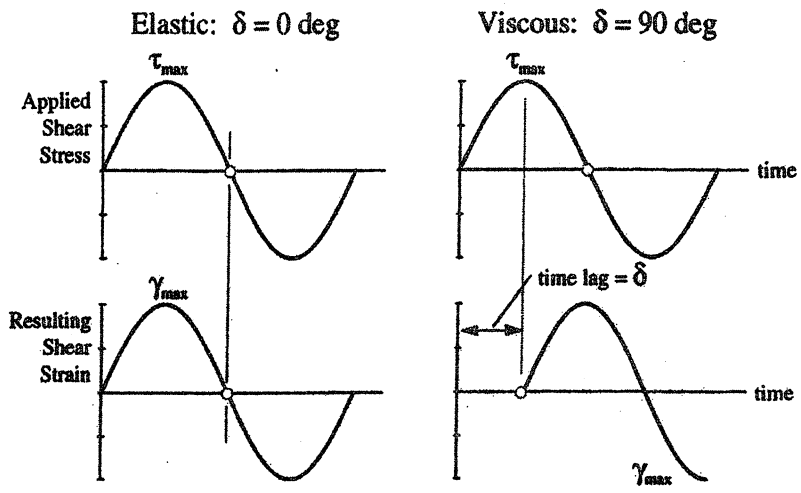


Figure 2–43. Stress–Strain Output of a Constant Stress Rheometer (*59*)

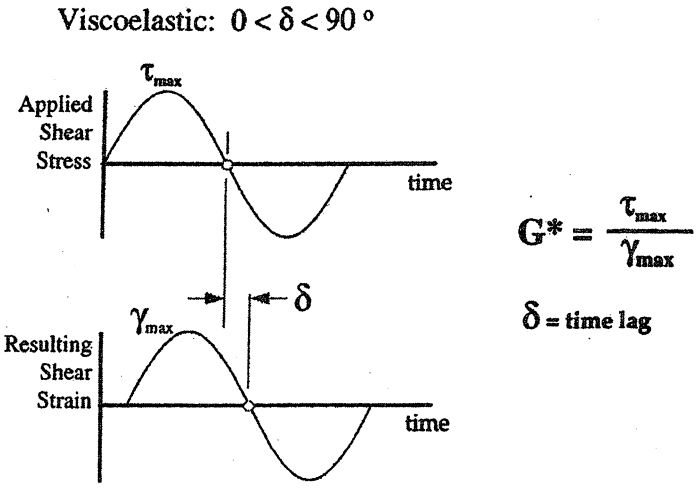


Figure 2–44. Stress–Strain Response of a Viscoelastic Material

The following formulas are used by the DSR software to calculate τ_{max} and γ_{max}.

$$\tau_{max} = 2T/\pi r^3$$

$$\gamma_{max} = \theta r/h$$

where,

T = maximum applied torque,
r = radius of binder specimen/plate (either 12.5 or 4 mm),
θ = deflection (rotation) angle,
h = specimen height (either 1 or 2 mm).

In constant stress rheometers used for PG binder testing, the constant stress used in testing should be such that the resulting shear strain is approximately 10–12 percent when relatively soft materials are tested at high temperatures (for example, unaged binders and RTFO aged binders), and about one percent when relatively hard materials (for example, PAV residues tested at moderate temperatures) are tested (*59*). The DSR automatically sets the constant stress levels for the desired percentage of strain. The DSR also computes and reports G* and δ upon completion of the test.

The DSR is used in the PG binder specification to measure the properties of the asphalt binder at high and intermediate pavement service temperatures. The actual test temperatures are determined based on the specified PG grade which is typically a function of the service temperatures in which the asphalt binder will function. A weather database of about 7500 North American weather stations over a 20–year period is used to determine the anticipated pavement service temperature at the project. The high temperature (maximum design temperature) is determined by averaging the maximum HMA pavement temperature over a seven–day period throughout the summer. Average values for successive seven–day periods are then determined and the largest seven–day average maximum pavement temperature is chosen as the maximum design temperature (*65*). The upper temperature grade is typically selected at this temperature; however, the grade is often bumped to a higher grade to enhance performance of pavements, such as interstate highways to be subjected to high traffic. The high temperature DSR testing is accomplished at this temperature. The intermediate testing

temperature is established as an approximate average of the seven–day average maximum and the minimum design temperatures. The minimum design temperature is chosen as the minimum pavement temperature expected to occur at the design location (*65*). This expected minimum is obtained from the low temperatures recorded in the database over a period of 20 years. Software is available to determine these HMA pavement design temperatures from the air temperatures in the weather database.

Physical property measurements such as G* and δ for a given asphalt binder are made three times with the DSR. The asphalt binder is tested in the DSR in its original (unaged), oven aged (RTFO residue), and PAV aged conditions. Original and RTFO aged asphalt binder samples are tested at the maximum design temperature to determine the binder's ability to resist rutting. PAV aged samples are tested at the intermediate design temperature to determine the asphalt binder's ability to resist fatigue cracking. A discussion of parameters used to quantify the asphalt binder's contribution in resisting rutting (permanent deformation) and fatigue cracking follows.

Rutting Parameter. For rutting resistance, a high complex modulus G* value and low phase angle δ are both desirable (*63*). The higher the G* value, the stiffer and thus the more resistant to rutting the asphalt binder will be. The lower the δ value is, the more elastic the asphalt binder.

The increased elasticity makes the asphalt binder more resistant to permanent deformation. The high temperature measurements for rutting are made on the original and TFO aged samples. The TFO aged samples represent the asphalt binder in the early stages of its life immediately after placement and before long–term oxidative aging begins. Since long term oxidative aging increases the stiffness of an asphalt binder, it continues to increase the binder's resistance to rutting. Testing of the asphalt binder prior to long term oxidative hardening, therefore, is critical with respect to rutting. The specification for DSR testing of the original asphalt binder was added as a safeguard against those asphalt

binders whose TFO aged residues do not indicate the actual aging during HMA production and placement in the field.

The SHRP researchers considered rutting as a stress controlled, cyclic loading phenomenon in determining the rutting parameter chosen for specification purposes. With each traffic loading cycle, work is being done to deform the HMA pavement surface. A part of this work is recovered by elastic rebound of the surface while some is dissipated in the form of permanent deformation and heat. In order to minimize permanent deformation (rutting), the amount of work dissipated during each loading cycle must be minimized. Mathematically, the work dissipated per loading cycle at a constant stress can be expressed as follows (*63*):

$$W_c = \pi \times \sigma_o^2 \times 1/G^*/\sin\delta$$

where

W_c = work dissipated per load cycle,
σ_o = stress applied during the load cycle,
G^* = complex modulus,
δ = phase angle.

This equation indicates that the work dissipated per loading cycle is inversely proportional to G*/sinδ. For this purpose, the G*/sinδ parameter was chosen as a Superpave asphalt binder specification. It can be seen from the preceding equation that the work dissipated per loading cycle can be decreased by either increasing the value of G* and/or decreasing the value of δ. This appears logical because increasing the G* of an asphalt binder makes it stiffer and thus more resistant to rutting. Also, decreasing the δ of an asphalt binder makes the asphalt binder more elastic, again increasing its resistance of rutting.

Fatigue Parameter. To determine the parameter for resistance to fatigue cracking for specification purposes, fatigue cracking was considered a strain controlled phenomenon. Fatigue cracking is typically considered a stress controlled phenomenon in thick HMA pavement

layers and a strain controlled phenomenon in thin HMA pavement layers. Since fatigue cracking is known to be more prevalent in thin pavements, the SHRP researchers assumed that it should be considered mainly a strain controlled phenomenon (*63*). Mathematically, the work dissipated per loading cycle at a constant strain can be expressed as follows:

$$W_c = \pi \times \varepsilon_o^2 (G^* \times \sin\delta)$$

where ε is the strain and the other variables are as previously described. This equation indicates that as G* and/or δ are increased, more work will be dissipated per traffic loading cycle. As G* decreases, the asphalt binder becomes less stiff and thus able to deform without building up large stresses. Additionally, asphalt binders with low δ values will be more elastic and thus can regain their original condition without dissipating work. Work can be dissipated in several different ways; heat dissipation, plastic flow, cracking, and crack propagation can all dissipate work. However, all of these contribute to pavement distresses. The lower the amount of energy dissipated per cycle, the lower the likelihood of fatigue cracking or any other damage phenomena to occur. The G*sinδ parameter was, therefore, chosen for Superpave specification purposes to limit the total amount of energy dissipated thereby minimizing fatigue cracking (*63*). PAV aged samples are tested in the DSR for determining the asphalt binder's ability to resist fatigue cracking. As the asphalt binder ages during its service life, it becomes stiffer thus causing it to become more susceptible to fatigue cracking. PAV aged samples are, therefore, used to test the asphalt binder to simulate the most critical in–service condition.

There are three separate specification requirements for the DSR test parameters. When the unaged asphalt binder is tested, the G*/sinδ value must be a minimum of 1 kPa. When the RTFO residue is tested for the same asphalt binder, the G*/sinδ value must be a minimum of 2.2 kPa. These two minimum limits were established for rutting purposes as previously discussed. The third DSR specification requirement is that the G*sin δ value of the PAV aged asphalt binder must be no more than

5000 kPa. This maximum specification limit was established to address fatigue cracking in HMA pavements.

Each of these specification limits was established by consensus of the Federal Highway Administration Asphalt Binder Expert Task Group (ETG). This group consists of persons from industry, specifying agencies, and academia. The group first established the minimum limit of 1 kPa for original asphalt binders. This was done by referencing the conventional viscosity data for paving asphalt cements at 60°C. This pavement service temperature was thought to be representative of a moderate climate where the use of AC–10 viscosity graded asphalt cement (viscosity of 1000 Poises) would be common. When unaged AC–10 asphalt cements are tested in the DSR at 10 rad/second, their G*/sinδ values are approximately 1 kPa. Therefore, the minimum limit of 1 kPa for the unaged asphalt binder was considered reasonable from a rutting standpoint because AC–10 asphalt cements were providing reasonable service in moderate climates.

The minimum limit of 2.2 kPa for the G*/sinδ value of RTFO aged asphalt binders was established based on the 1 kPa value of the unaged asphalt binder. RTFO test data on asphalt cements indicated that, on the average, the aging index (viscosity after RTFO/viscosity before RTFO) for asphalt cements ranged from 2 to 2.5. In other words, asphalt binders become from 2 to 2.5 times stiffer when aged in the RTFO. Therefore, the average of the range, 2.2 was used to establish the 2.2 kPa minimum limit for RTFO aged asphalt binders.

The original specification for fatigue cracking called for a maximum limit of 3000 kPa. However, when the test results of the 42 asphalt binders used to set the limits were investigated, half would not meet the 3000 kPa maximum criterion. The ETG, therefore, deemed the 3000 kPa maximum limit to be too restrictive. It was decided to use a higher value of 5000 kPa until field validation data could be gathered and analyzed. Since about 15 percent of the 42 asphalt binders would not meet this limit, the value of 5000 kPa appeared reasonable at the time.

Bending Beam Rheometer (BBR). The Bending Beam rheometer (BBR) tests asphalt binders at low pavement service temperatures to determine the binder's propensity to thermal cracking. Thermal cracking of HMA pavements results when the temperature drops rapidly at cold temperatures. As the pavement contracts, stresses begin to build up within the HMA pavement layers. If the contraction occurs very rapidly the stresses can build and eventually exceed the stress relaxation ability of the HMA pavement. When this occurs, the pavement develops cracking as a way to relieve stress. Thermal cracking can result from one thermal cycle when a critically low temperature is encountered, or from thermal cycling where the temperature cycles up and down, but always remains above the critical low temperature. Cracking caused by a single thermal cycle is related to the asphalt binder stiffness at the temperature at which cracking occurs. This temperature is referred to as the limiting stiffness temperature. The limiting stiffness temperature is, therefore, defined as the pavement service temperature at which a certain stiffness value is reached after a specified loading time.

A device to determine the properties and response of asphalt binders at low temperatures was initially developed in the 1980s (*66*). This equipment was modified and updated as part of the SHRP binder research. The resulting machine was named the Bending Beam Rheometer (BBR). The testing procedure is given in AASHTO TP1.

The BBR uses a transient creep load, applied in the bending mode, to load an asphalt beam specimen held at a constant low temperature. The temperature is maintained by a fluid bath filled with a mixture of ethylene glycol, methanol, and water. Both the specimen geometry and the loading technique closely resemble ASTM D 790, Test Methods for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials (*67*). The BBR consists of a loading mechanism, temperature control bath, and data acquisition system as shown in the schematic in Figure 2–45. The dimensions of the beams were selected in conformity with ASTM D 790 so that the elementary Bernoulli–Euler theory of bending prismatic beams would apply (*62*). The BBR uses a blunt–nosed shaft to apply a midpoint load to the asphalt

beam resting on two end supports. The load cell mounted on the loading shaft is enclosed in an air bearing to eliminate frictional resistance. A transducer affixed to the shaft monitors deflections of the beam. Loads are applied by pneumatic pressure. The data acquisition system records the load and deflection test results and calculates two parameters: (a) creep stiffness, S(t) which is a measure of how the asphalt binder resists the constant (creep) loading, and (b) m–value which is a measure of the rate at which the creep stiffness changes with loading time.

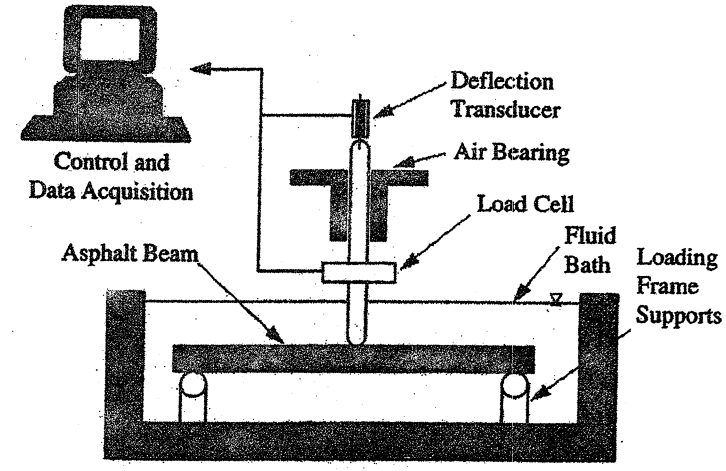


Figure 2–45. Schematic of Bending Beam Rheometer (*59*)

Asphalt binder beams measuring 125 mm in length, 6.25 mm in width, and 12.5 mm is height are formed by pouring heated binder into aluminum or silicone molds. The beam specimens are removed from the mold after cooling the mold assembly in a freezer for 5–10 minutes. The beam is then kept in the test bath for 60±5 minutes before testing. After pre–loading procedures (*60*), a 100–gram load is applied to the beam for a total of 240 seconds. The deflection of the beam is recorded during this loading period as shown in Figure 2–46. Load and deflection versus time plots are also generated on the computer screen. After 240 seconds, the

BBR software performs all calculations to determine creep stiffness and m–value as follows.

Classic beam analysis theory is used to calculate the creep stiffness of the asphalt binder beam at 60 seconds loading time (*59*):

$$S(t) = PL^3/4bh^3\delta(t)$$

where,

$S(t)$ = creep stiffness at time, t = 60 seconds
P = applied constant load, 100 g (980 mN)
L = distance between beam supports, 102 mm
b = beam width, 12.5 mm
h = beam thickness, 6.25 mm
$\delta(t)$ = deflection at time, t = 60 seconds

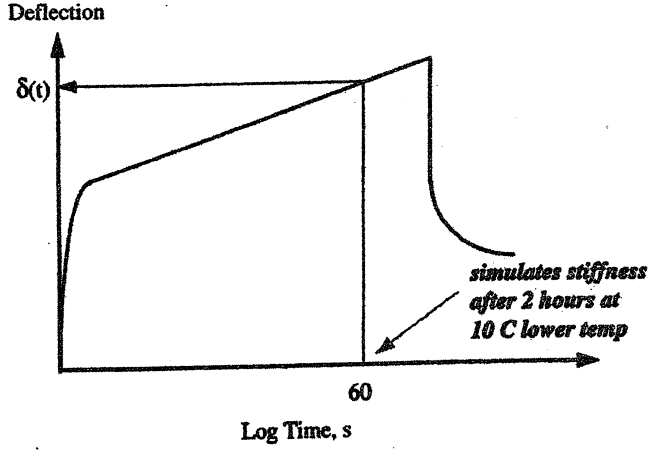


Figure 2–46. Deflection versus Time for BBR (*59*)

Although creep stiffness is desired at the minimum HMA pavement design temperature after two hours loading time based on past research, the testing time will be too long. It was determined from time temperature superposition principle discussed later that by raising the test temperature 10°C, an equal creep stiffness is obtained after a 60–second loading time which is more realistic for routine testing.

The second parameter obtained by the BBR is the m–value. The m–value indicates the rate of change of the stiffness, S(t), with loading time as shown in Figure 2–47. In order words, the m–value is the slope of the log creep stiffness versus log time curve at any time, t. The PG binder specification requires that the m–value be greater than or equal to 0.300 when measured at 60 seconds (*59*).

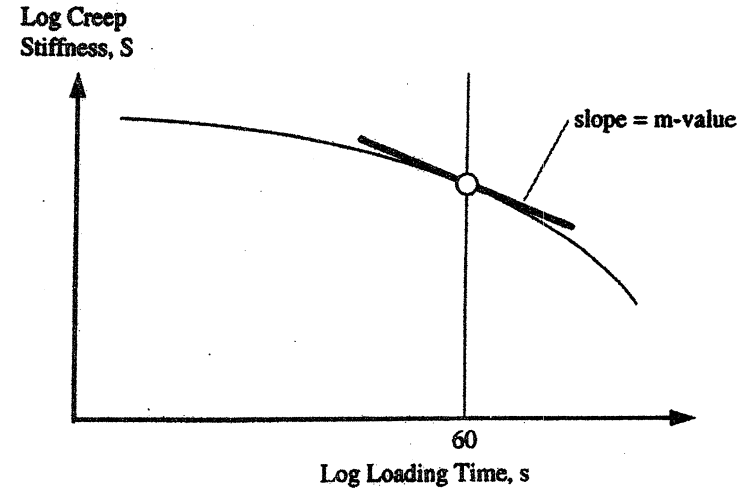


Figure 2–47. m–value from Bending Beam Rheometer (*59*)

A discussion of the BBR test development and specification requirement follows.

In developing the BBR test method to determine the rheological parameters that affect thermal cracking, the SHRP researchers performed a series of tests to determine the critical factors involved such as time and temperature dependency, physical hardening and conditioning time, and the effect of load levels (*59*).

To develop performance based specifications, the laboratory test intended for thermal cracking specification purposes should simulate field conditions. However, most researchers (*65*, *48*) have correlated thermal cracking of in–service HMA pavements with asphalt binder stiffness values estimated at loading times of 3,600–20,000 seconds (1–

5.5 hours). These loading times are rather excessive for a laboratory test procedure used for specification purposes. Therefore, the time–temperature superposition principle was used to shorten the laboratory test time. Using this principle, results are obtained for a shorter loading time using one or more temperatures and the response at the longer loading time estimated. Preliminary tests showed that a loading time of 240 seconds was reasonable. A series of tests performed at different temperatures indicated that at the lowest pavement temperatures, the time–temperature superposition shift functions for many asphalt binders are similar regardless of loading time. This indication led to the establishment of a single offset temperature for estimating the response at longer loading times. It was determined that an offset of 10°C above the lowest pavement temperature could be used to equate the BBR stiffness at 60 seconds loading time to the asphalt binder stiffness at 7,200 seconds loading time in the field at the lowest pavement temperature (*62*). A limiting stiffness based on 2 hours (7,200 seconds) was selected as one of the criteria based upon the literature (*65*).

The BBR loads the beams for 240 seconds and reports the stiffness values at loading times of 8, 15, 30, 60, 120, and 240 seconds. These values were chosen because they are fairly equally spaced on a logarithmic time scale (*62*). These data points, along with the following equation, are used to determine the shape of the master stiffness (creep compliance) curve for the asphalt binder.

$$S(t) = A + B\log(t) + C[\log(t)]^2$$

where,

$S(t)$ = the asphalt binder stiffness,
t = time,
A, B, and C = constants.

Since the time dependency of asphalt binders varies, the shape of the master stiffness curve as well as the stiffness at 2 hours loading time are important. Therefore, the slope of the master stiffness curve is also used for specification purposes (*65*). The stiffness, S(t), is a measure of the

thermal stresses developed in the HMA pavement as a result of thermal contraction. The slope of the stiffness curve, m, is a measure of the rate of stress relaxation by asphalt binder flow as discussed previously (*62*). The effect of these two specification parameters, S(t) and m–value, on thermal cracking is analogous to the effect of G* and δ on rutting and fatigue cracking. As S(t) increases, the thermal stresses developed in the pavement due to thermal shrinking also increase, and thermal cracking becomes more likely. On the other hand, as the m–value decreases, the rate of stress relaxation decreases. In other words, as the slope of the asphalt binder stiffness curve flattens, the ability of the HMA pavement to relieve thermal stresses by flow decreases. This again would increase the propensity of thermal cracking in the pavement. Accordingly, the PG binder specification requires a maximum limit of S(t) at 60 seconds and a minimum m–value. The numerical limits for these two parameters were chosen as S(60)=300 MPa maximum, and m=0.300 minimum. These limits were established based on data from previous studies as well as the data obtained by SHRP (*62*). A report (*68*) based on six asphalt cement binders used in Pennsylvania in 1976 (*48*) has offered some insight into the field validation of these two parameters. The six test sections, which used AC–20 asphalt cements from different sources, were monitored for thermal cracking for a period of 7 years. Samples of the original asphalt binders were retained and tested in 1995 using the Superpave binder equipment. The results indicated that the S and m–values chosen in Superpave binder specifications were generally reasonable. The low temperature behavior of all but one of the AC–20 asphalt cements could be explained by these values (*68*).

Physical hardening is a phenomenon which was observed by the SHRP researchers. The phenomenon is believed to be caused by delayed volume shrinkage and results in significant time–dependent hardening (*69*). How this phenomenon affects the HMA pavement performance is not known. Therefore, no specification limit could be established for physical hardening (*65*). However, the results of the physical hardening that occur over 24 hours of isothermal storage can be reported according to Superpave binder specification. To minimize the effect of physical

hardening on the BBR test data, a standard conditioning time of 60±5 minutes was selected as mentioned before.

For the BBR test results to be valid, the measurements must be made in the linear viscoelastic region. A small experiment which involved testing replicate beams with loads ranging from 20–150 grams was conducted by SHRP researchers to ensure that this was the case. It was concluded that in this load range, stress and strain were independent. Based on the test results, the researchers established a standard load of 100 grams (*62*).

Direct Tension Tester (DDT). There is a fairly good relationship between the stiffness of asphalt binders and the amount of stretching (ductility) they undergo before breaking at low temperatures. Typically, softer asphalt binders are more ductile and stiffer asphalt binders are more brittle (those that break without much stretching). Creep stiffness of the asphalt binders as measured by the BBR is not adequate to completely characterize the low temperature behavior of the asphalt binders in terms of thermal cracking. There are asphalt binders, particularly some modified asphalt binders, which may have high creep stiffness (more than 300 MPa) but do not crack because they can stretch further before breaking. Therefore, the DDT was developed to address these stiff, ductile asphalt binders. The test procedure is given in AASHTO TP3.

The DTT measures the low temperature ultimate tensile strain of the asphalt binders, typically in the 0° to –36°C range. Like the BBR test, the asphalt binder is subjected to RTFO and PAV aging before the DTT test. A small dog–bone shaped specimen is loaded in tension at a constant rate (1 mm/min.) as shown in Figure 2–48 until it breaks. The failure strain in the specimen is the change in length (ΔL) divided by the effective gauge length (L_e). The failure strain in the DTT corresponds to the stress where the load on the specimen reaches its maximum value, and not necessarily the load when the specimen breaks.

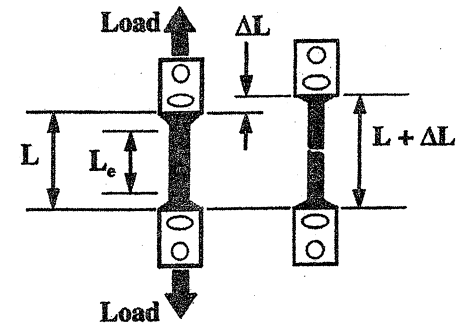


Figure 2–48. Direct Tension Test (*59*)

Superpave Asphalt Binder Specification

The PG asphalt binder specification is provided in AASHTO MP1–93. The physical properties included in the specification are measured by the test equipment described earlier. As mentioned before, the physical properties remain constant for all performance grades (PG), but the temperature at which these properties must be achieved varies from grade to grade depending on the climate in which the asphalt binder is expected to perform (Figure 2–49). For example, a PG 64–28 grade is intended for use in an environment where an average seven–day maximum pavement temperature of 64°C and a minimum pavement design temperature of –28°C, are likely to be experienced. As stated earlier many agencies will increase the high temperature PG grade for asphalt pavements to be subjected to high traffic volumes.

Since the Superpave asphalt binder specification is meant to be performance based, it addresses three primary performance parameters of HMA pavements: permanent deformation (rutting), fatigue cracking, and low temperature (thermal) cracking. In addition, the specification addresses concerns related to safety, pumping and handling, and excessive aging of the asphalt binders. A discussion of these parameters addressed in the Superpave binder specification follows.

Performance Grade	PG 52							PG 58		
	-10	-16	-22	-28	-34	-40	-46	-16	-22	-28
Average 7-day Maximum Pavement Design Temp, C	<52							<58		
Minimum Pavement Design Temperature, C	>-10	>-16	>-22	>-28	>-34	>-40	>-46	>-16	>-22	>-28
	Original Binder									
Flash Point Temp, T48: Minimum, C	230									
Viscosity, ASTM D 4402:[b] Maximum, 3 Pa-s (3000 cP) Test Temp, C	135									
Dynamic Shear, TP5:[c] G*/sin δ, Minimum, 1.00 kPa Test Temp @ 10 rad/sec, C	52							58		

Spec Requirement Remains Constant

Test Temperature Changes

Figure 2–49. PG Binder Specification Example (*59*)

Permanent Deformation (Rutting). The Superpave binder specification uses a rutting factor, G*/sinδ, which is a measure of asphalt binder's stiffness or rut resistance at high pavement service temperatures. The significance of this factor was addressed when discussing the DSR. As shown in Figure 2–50, the rutting factor must be at least 1.00 kPa for the original asphalt binder and a minimum of 2.20 kPa for the RTFO aged asphalt binder when tested by DSR at the maximum pavement design temperature.

Performance Grade
Average 7-day Maximum Pavement Design Temperature, C
Minimum Pavement Design Temperature, C
Flash Point Temp, T48: minumum C
Viscosity, ASTM D 4402:[b] Maximum, 3 Pa-s (3000 cP), Test Temp, C
Dynamic Shear,TP5:[c] G*/sin δ, Minimum, 1.00 kPa Test Temperature @ 10 rad/s, C
Rolling Thin Film Oven (T240)
Mass Loss, Maximum, %
Dynamic Shear, TP5: G*/sin δ, Minimum, 2.20 kPa Test Temp @ 10 rad/sec, C

Spec Requirements to Control Rutting

Figure 2–50. PG Specification Rutting Factor Requirements (*59*)

Fatigue Cracking. The specification uses a fatigue factor, G* sinδ, which represents asphalt binder's resistance to fatigue cracking. As shown in Figure 2–51, the specification has a maximum limit of 5000 kPa for G* sinδ for the asphalt binder subjected to RTFO and PAV aging, and tested at intermediate pavement service temperature. For example, the intermediate pavement service temperature for a PG 64–28 grade is 22°C.

PAV Aging Temp, C
Dynamic Shear, TP5: G*sin δ, Maximum, 5000 kPa Test Temp @ 10 rad/sec, C
Physical Hardening[e]
Creep Stiffness, TP1:[f] S, Maximum, 300 MPa m-value, Minimum, 0.300 Test Temp, @60 sec, C
Direct Tension, TP3:[f] Failure Strain, Minimum, 1.0% Test Temp @ 1.0 mm/min, C

Specification requirement to control fatigue cracking

Figure 2–51. Superpave Specification Fatigue Cracking Factor Requirements (*59*)

air temperature for each year and the means and standard deviation of the average one–day minimum air temperatures were also determined.

Since the design temperatures for selecting PG grade are based on pavement temperatures rather than air temperatures, the Superpave software converts the air temperatures to pavement temperatures using theoretical analyses of actual conditions performed with models (*70*). At the present time (1996), the Superpave criteria considers the minimum pavement temperature equal to the minimum air temperature. This is likely to change in the future based on field data. It is believed that the minimum pavement temperature is generally higher (warmer) than the minimum air temperature.

The Superpave software also allows the designer to use different levels of reliability concerning the design pavement temperatures. Reliability has been defined as the percent probability in a single year that the actual temperature (one–day low or seven–day average high) will not exceed the corresponding design temperatures. Consider the example of Topeka, Kansas as shown in Figure 2–53 (*59*). It has a mean seven–day maximum air temperature of 36°C and a standard deviation of 2°C. According to the frequency distribution, there is a 50% probability in an average year that the seven–day maximum air temperature will exceed 36°C. However, the chance that the temperature will exceed 40°C is only two percent. In other words, a design high air temperature of 40°C provides 98 percent reliability.

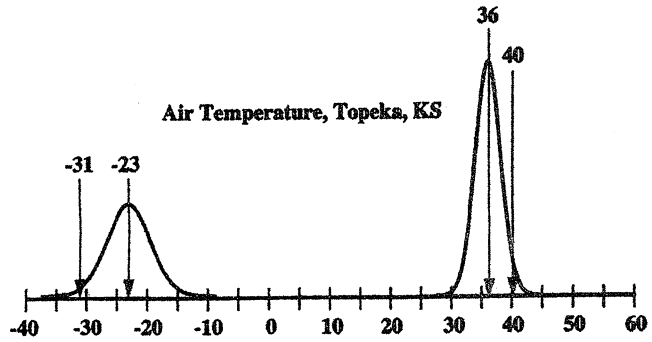


Figure 2–53. Distribution of High and Low Design Air Temperatures for Topeka, KS (*59*)

Similarly, the average 1–day coldest temperature is –23°C with a standard deviation of 4°C which means a very cold winter could have a minimum air temperature of –31°C at 98% probability.

The high and low air temperatures for Topeka, Kansas are converted to pavement temperatures by the Superpave software as shown in Figure 2–54. The high pavement temperature is calculated 20 mm below the pavement surface and the low temperature is calculated at the surface for the wearing course in this example. Figure 2–55 shows how PG grades are selected at 50 and 98% minimum reliability levels. For at least 50% minimum reliability, the high temperature grade must be PG 58. Since there is no PG 56 grade it has to be "rounded up" to PG 58, which actually results in a probability of about 85 percent rather than 50 percent. On the other hand, the low temperature grade for at least 50% reliability must be –28°C which gives almost 90% reliability rather than 50%. For 98 percent reliability, a PG 64 grade is needed for high temperature and a PG–34 grade for low temperature resulting in a PG 64–34 grade.

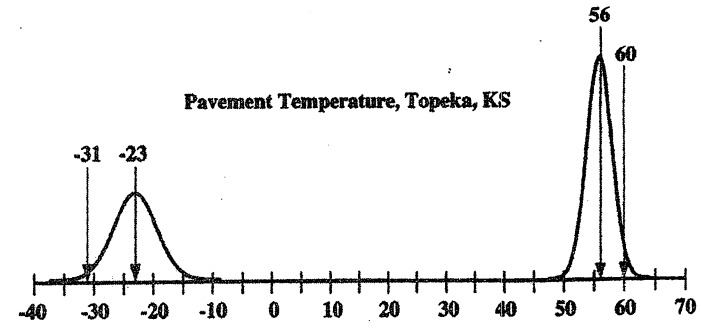


Figure 2–54. Distribution of High and Low Design Pavement Temperatures for Topeka, KS (*59*)

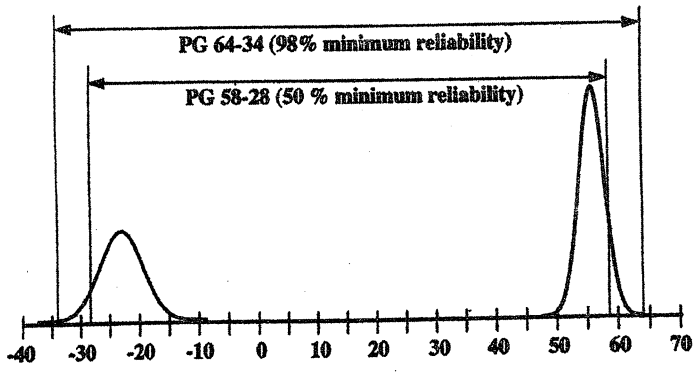


Figure 2–55. Binder Grades for Topeka, KS (*59*)

The preceding binder selection process based on climate only assumes that the HMA pavements will be subjected to fast moving traffic. The DSR tests to determine the rutting factor $G^*/\sin\delta$ are conducted with a loading rate of 10 radians per second which corresponds to a traffic speed of about 90 kilometers per hour. However, significantly slower loading rates are experienced by HMA pavements at some locations such as intersections and toll booths. There are cases such as bus stops where loads are also stationary. In such cases of slower loading rates an asphalt binder must exhibit a higher stiffness to minimize rutting. To meet these situations, the high temperature grade should be increased by one grade for slow transient loads and by two grades for stationary loads. For example, if a PG 64–22 grade was selected based on climate for fast moving traffic, a PG 70–22 grade would be used for slow transient loads, and a PG 76–22 grade would be used for stationary loads. In practice, many state DOTs actually bump the recommended grade by 2 grades for high traffic volumes roads to ensure better resistance to rutting. The selected low temperature grade is not affected by the loading rate. Since the highest possible pavement temperature in North America is about 70°C, it was necessary to add two high temperature grades, PG 76 and PG 82, to accommodate slow traffic loading rates (*59*).

Most state highway agencies have adopted the Superpave asphalt binder system.

ASPHALT CHEMISTRY

The importance of asphalt chemical composition, although not well understood, cannot be disputed. The performance of asphalt as a binder in HMA pavements is determined by its physical properties (discussed earlier in the chapter) which in turn are determined directly by chemical composition. An understanding of the chemical factors affecting physical properties is thus fundamental to an understanding of the factors that control asphalt performance. This section on asphalt chemistry is largely adapted from the papers authored by Petersen (*71*) and Halstead (*72*). An attempt has been made to avoid detailed discussion of complex chemical phenomena and to minimize the use of terminology unfamiliar to civil engineers.

Origin of Asphalt

Asphalt cement is refined from the crude petroleum which is primarily formed by nature from plant life. The process of transformation from plant life to crude oil occurs over millions of years under varied temperature and pressure conditions. Although all petroleums are basically hydrocarbons (chemical combinations of carbon and hydrogen), the amount and nature of hydrocarbons varies from crude to crude. Since the asphalt cement is obtained by distillation from the crude, its chemical composition and properties also vary from source to source.

Elemental Composition and Molecular Structure

Carbon and hydrogen are the principal elements present in asphalt cement molecules. Sulfur is the next most abundant element. Nitrogen and oxygen are usually present in very small amounts. Heavy metals such as vanadium and nickel may also be present in trace amounts.

Elemental analyses of four representative petroleum asphalt cements are given in Table 2–4.

Table 2–4. Elemental Analyses of Representative Petroleum Asphalts (71)

Elements	B–2959 Mexican Blend	B–3036 Arkansas–Louisiana	B–3051 Boscan	B–3602 California
Carbon, percent	83.77	85.78	82.90	86.77
Hydrogen, percent	9.91	10.19	10.45	10.93
Nitrogen, percent	0.28	0.26	0.78	1.10
Sulfur, percent	5.25	3.41	5.43	0.99
Oxygen, percent	0.77	0.36	0.29	0.20
Vanadium, ppm	180	7	1380	4
Nickel, ppm	22	0.4	109	6

Most asphalt molecules consisting of carbon and hydrogen contain one or more of the following so–called heteroatoms: sulfur, nitrogen and oxygen. The type of molecular structure is more important than the total amount of each element. Since the petroleum is transformed from diverse living organic matter under different conditions, asphalt cements have extremely diverse molecular structures depending on the crude source.

Asphalt cements contain a combination of the following three arrangements by which the carbon atoms are linked with each other:

1. Straight or branched chains. Such asphalt cements are generally called "aliphatic" or "paraffinic" types.
2. Simple or complex saturated rings. "Saturated" means the highest possible hydrogen/carbon ratio in the asphalt molecules. These asphalt cements are usually referred to as "naphthenic" types.
3. One or more stable six–carbon condensed, unsaturated ring structures. These asphalt cements are called "aromatic" types. Benzene and naphthalene have such structures.

Functional or Polar Groups

As mentioned earlier, heteroatoms (sulfur, nitrogen and oxygen) are attached to carbon atoms in asphalt molecules in different configurations and in form of different compounds. These configurations (molecules) are polar because there is an imbalance of electrochemical forces within the molecule which produces a dipole. Therefore, each polar molecule has electropositive and electronegative characteristics similar to a magnet which has north and south poles. Since like charges repel each other and unlike charges attract each other, molecular interactions are induced in asphalt cements which strongly influence their physical or "engineering" properties and performance. These configurations of heteroatoms thus impart functionality and polarity to asphalt molecules and are therefore called functional or polar groups. Functionality (presence of function groups) relates to how the asphalt molecules interact with each other or with surfaces and/or molecules of other materials. For example, selective adsorption of functional or polar groups by aggregate surfaces is important to understand the stripping phenomenon. Although present in small amounts, these polar groups vary in characteristics from asphalt to asphalt, and have a large effect on the performance of asphalt cement from different sources.

Also, the non–polar components of asphalt cement, which act as solvents or dispersants for the polar or functional groups, play a major role in determining the effect that the polar groups will have on the physical and aging properties of the asphalt cement.

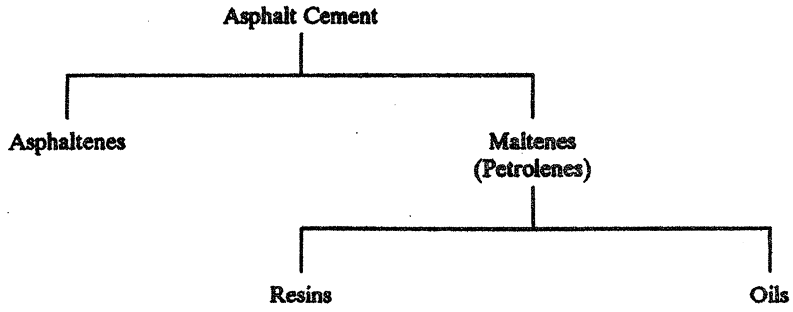
The types of sulfur, nitrogen, and oxygen compounds which have been identified in asphalt cements are given in Table 2–5.

Table 2–5 Sulfur, Nitrogen, and Oxygen Compounds

Sulfur Compounds	Polysulfides
	Sulfides
	Thiols
	Thiophenes
Nitrogen Compounds	Pyridinic
	Pyrrole, indole, carbazole
	Porphyrins
Oxygen Compounds	Carboxylic acids/naphthenic acids
	Phenols
	Ketones
	Esters
	Ethers
	Anhydrides

Conceptual Compositional Model

According to the most simple and generally accepted concept of asphalt composition, asphalt cements are considered to be made up of asphaltenes, resins, and oils as shown in the following chart:



Asphaltenes are insoluble (or precipitated) when the asphalt cement is dissolved in a nonpolar solvent such as pentane, hexane, or heptane. The component which is dissolved is called "maltenes" (or "petrolenes"), and it is comprised of resins and oils.

Asphaltenes. Asphaltenes are generally dark brown, friable solids. The type of nonpolar solvent used to precipitate the asphaltenes affects the determination of its total amount in the asphalt cement. Higher amounts are precipitated by n–pentane than by n–heptane because the number of carbon atoms in n–pentane is less than that in n–heptane. Asphaltenes are the most complex components with the highest polarity. Therefore, they have a very high tendency to interact and associate. They are mixtures of many compounds that have a strong tendency to associate in conglomerates. Asphaltenes play a major role as the viscosity–building ("bodying") component of asphalt cements. Studies have shown that the amounts and characteristics of asphaltenes vary considerably from asphalt to asphalt. Low asphaltene content (less than about 10 percent) or weakly associating asphaltenes have been linked with tenderness in HMA by some researchers.

Resins. Resins are generally dark and semi–solid or solid in character. They are fluid when heated and become brittle when cold. They work as agents that disperse (or "peptize") the asphaltenes throughout the oils to provide a homogeneous liquid. They are soluble in n–pentane, but adsorb on fuller's earth or alumina. On oxidation resins yield asphaltene type molecules.

Oils. Oils are usually colorless or white liquids. They are soluble in most solvents. They have paraffinic and naphthenic structures with no oxygen and nitrogen usually present. On oxidation they yield asphaltene and resin molecules.

Asphalt as a Colloidal System

Asphalt cement is not a true solution, but is considered a colloidal or micellar system. Nellensteyn (*73*) first recognized its colloidal nature by stating that it has a dispersion of micelles in an oily medium. The relative amounts and characteristics of asphaltenes, resins, and oils present in an asphalt cement influence its physical properties and performance in a HMA mixture. These influences make the asphalt act as a "sol," "sol–gel," or "gel."

"Sol:" "Sol" asphalt cements typify a system in which the resins keep the asphaltenes highly "peptized" (or dispersed) in the oily phase. "Sol" asphalt cements largely exhibit Newtonian flow characteristics. According to Rostler (*74*), nitrogen bases in asphalt cement are primarily responsible for its "sol" characteristics.

"Sol–Gel:" It is intermediate between "sol" and "gel."

"Gel:" "Gel" asphalt cements typify a system in which resins are not very effective in "peptizing" asphaltenes. Excessive presence of paraffins in relation to nitrogen bases also tends to reduce solubility, leading to increased "gel" characteristics, and suggesting increased separation of the dispersed and dispersing phases (*74*). "Gel" asphalt cements exhibit largely non–Newtonian behavior.

The degree to which an asphalt cement acts as a "sol" or as a "gel" influences its rheological properties, aging characteristics, and its performance in a HMA mixture.

The compatibility and relationships of different components in the microscopically homogeneous mixture controls the overall behavior of asphalt cement rather than the quantitative amount of any single component according to Petersen (*71*). Various components of asphalt cement interact to form a balance or compatible system, which gives asphalt cement its unique viscoelastic properties as a binder. It has long been recognized that this compatibility, or lack of compatibility, is

related to asphalt quality. Oliensis's spot test (*75*) was an attempt to determine the "stability" of the asphalt components when dissolved in a solvent. Many agencies adopted the Oliensis spot test as a specification test for homogeneity of asphalt cements and therefore as an indicator of asphalt cement's durability in service. However, as was shown by Heithaus and Fink (*76*), the spot test did not distinguish between "good" and "bad" asphalt cements. This test did serve a good purpose when first introduced because it identified some nondurable asphalt cements obtained from the cracking of petroleum in certain refining processes.

Analytical Procedures

The two most frequently used fractionation methods are: (1) Rostler and Sternberg's chemical precipitation method, and (2) Corbett's selective adsorption–desorption (chromatographic) method. However, in all cases the "generic" fractions are still complex mixtures and not defined chemical species; thus the same generic fraction from different asphalt cements may have a quantitatively different effect on the physical properties.

Chemical Precipitation Method. It is only an analytical method; it does not physically separate the components in asphalt cement. Figure 2–56 gives the flow chart for this method developed by Rostler, Sternberg, and White (*77, 78*), who attempted to identify and quantify five components in asphalt cement. Essentially, the asphaltenes are separated (precipitated) by n–pentane and the solution of maltenes is sequentially treated with increasing strengths of sulfuric acid (H_2SO_4) to measure the amounts of four classes of molecular (or components) present in the maltenes. The following five components are quantified in this method:

1. *Asphaltenes (A):*

Asphaltenes are precipitated by n–pentane. They are considered as "bodying" agents in asphalt cement.

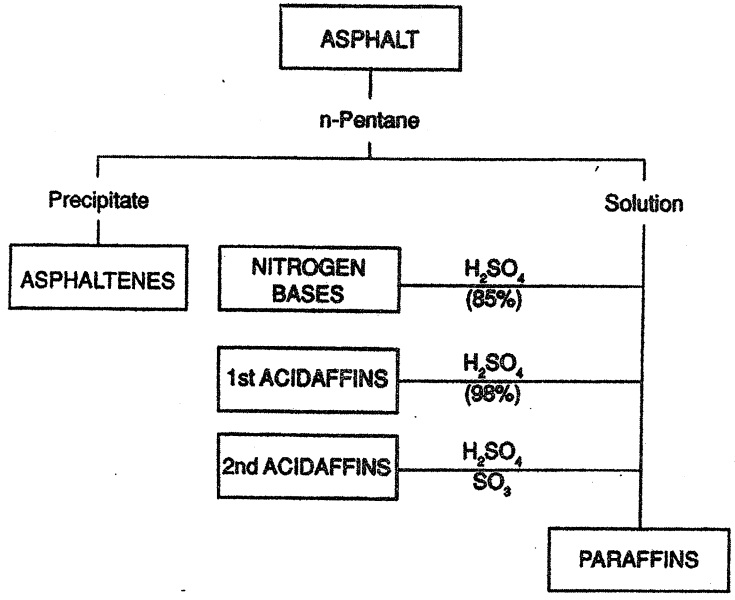


Figure 2–56. Chemical Precipitation Method (after Rostler and Sternberg, 77)

2. *Nitrogen Bases (N):*

85 percent sulfuric acid is used to remove or precipitate polar nitrogen compounds (quite aromatic in nature) from the maltene solution. This fraction is called "nitrogen bases" and is considered to be the peptizer for the strongly associated asphaltenes in the solvating phase.

3. *First Acidaffins (A1):*

98 percent sulfuric acid is then used to precipitate another aromatic component low in nitrogen. Rostler called it "first acidaffins" which are considered solvents for the peptized asphaltenes.

4. *Second Acidaffins (A2):*

Fuming sulfuric acid containing 30 percent SO_3 is then used to precipitate a less reactive and less aromatic component of the maltenes called "second acidaffins." This component is also believed to be the solvent for peptized asphaltenes.

5. *Paraffins (P):*

This is the final and oily constituent of the maltenes which is nonreactive to the fuming acid. According to Rostler, paraffins are believed to act as "gelling agents."

Many asphalt technologists with an engineering background have difficulty relating the results from this analytical procedure using sulfuric acid of different strengths to the performance of asphalt cement. However, Rostler has suggested the following relationships concerning the quality and durability of asphalt cement:

$$Compatibility\ Ratio = \frac{N}{P}$$

A compatibility ratio of more than 0.5 is considered good.

$$Durability\ Parameter = \frac{N + A1}{P + A2} = \frac{most\ reactive}{least\ reactive}$$

A durability parameter of less than 0.40 is considered poor, and more than 1.00 is considered good.

Selective Adsorption–Desorption Method. This method (ASTM D4124) developed by Corbett uses selective adsorption–desorption procedures (Figure 2–57) and actually separates the components virtually unaltered for further analysis (79). The most polar and least soluble asphalt cement components (asphaltenes) are first precipitated by n–

heptane which is a nonpolar solvent. The heptane solution of the remaining maltene fraction is then introduced into a chromatographic column. Alumina in the column adsorbs the remaining three components. Different solvents of increasing polarity are then used to sequentially desorb (wash out) fractions of increasing polarity. Corbett's method separates and quantifies the following four components:

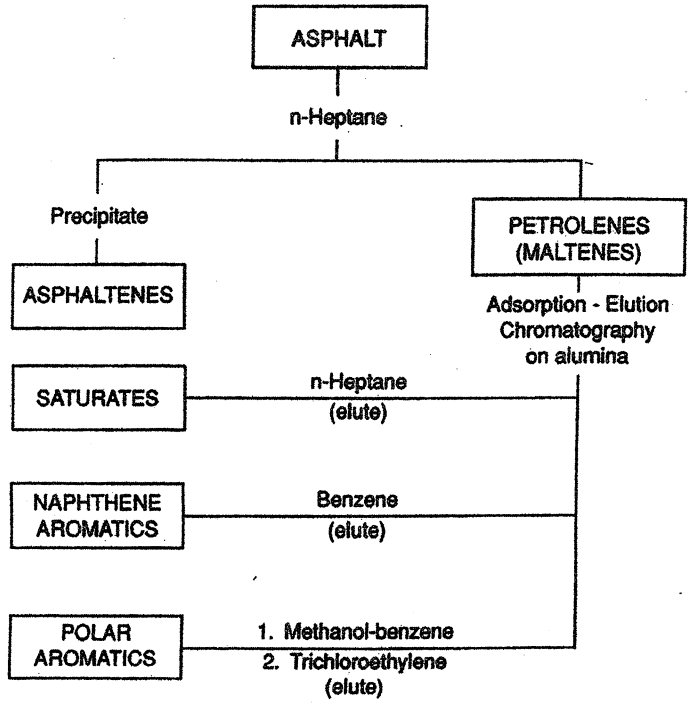


Figure 2–57. Selective Adsorption–Desorption Method (after Corbett, 79)

1. *Asphaltenes (A):*

Asphaltenes precipitated by n–heptane are most polar and act as the "bodying" fraction of asphalt cement as discussed earlier. They are solid or semi–solid at ambient temperature.

2. *Saturates (S):*

Saturates are the first fraction to emerge from the column when eluted (desorbed) with n–heptane. This fraction is not adsorbed by the alumina in the column because it lacks polar chemical functional groups. Saturates are equivalent to paraffins in the Rostler analysis. Often sulfur is found incorporated in molecules of this fraction. Saturates are liquid at ambient temperatures and hardly change with time. They have a negative contribution to temperature susceptibility of asphalt cement.

3. *Naphthene Aromatics (NA):*

Naphthene aromatics emerge as the second fraction when eluted with a more polar aromatic solvent such as benzene or toluene. Their molecular structure is comprised of condensed nonaromatic and aromatic ring systems, and may contain the heteroatoms oxygen, nitrogen and sulfur. They are liquid at ambient temperatures. They are considered to be the softening component in asphalt cement. They are also the aging fraction in asphalt cement.

4. *Polar Aromatics (PA):*

They are the final fraction to emerge from the column when eluted with a highly polar mixture of alcohol and benzene (or toluene). Alcohol helps to debond this strongly adsorbed and most polar component of the maltenes from the absorbent (alumina). Its molecular structure comprises a higher percentage of condensed aromatic ring systems and functional groups containing heteroatoms than the other fractions of maltenes. It is solid or semi–solid at ambient temperatures. It has been related to ductility of asphalt cement and is also one of the aging fractions in asphalt cement.

Molecular Size Distribution. High pressure liquid chromatography (HPLC), or high pressure gel permeation chromatography (HP–GPC), has been used to determine the molecular size distribution in asphalt cement. It is analogous to gradation (or sieve analysis) of mineral aggregates.

A chromatogram of the relative amounts of large, medium, and small molecules in an asphalt cement is obtained by this procedure. Work with this technique has been reported by Jennings at Montana State University (*80*). Essentially, a solution of asphalt cement (tetrahydrofuran or THF has been used as solvent) is passed through gel permeation columns. The system permits largest molecules to pass quickly through the columns but successively retards the progress of the smaller molecules which are slowed in their movement through the gel as they move in and out of the pores of the gel while moving through the column.

A detector and recorder produces a chromatogram showing the relative amounts of molecules being eluted at different times. Jennings has classified the molecules eluted during the first third of the elusion period as large molecular size (LMS), those eluted during the second third of the period as medium molecular size (MMS), and those eluted during the final third of the period as small molecular size (SMS).

Figure 2–58 illustrates two chromatograms obtained by this method on two asphalt cements, A and B. These asphalt cements have significantly different molecular size distributions, and therefore significantly different performances would generally be expected.

Jennings (*80*) compared the relative amounts of LMS, MMS, and SMS in asphalt cements recovered from poorly performing HMA pavements with those from good HMA pavements. Results obtained from Montana roads indicate greater relative amounts of LMS are associated with poor performance. It is possible that a proper balance between the relative amounts of LMS, MMS, and SMS in an asphalt cement is desirable for its durability and ability to perform as a binder. More recent work in the Strategic Highway Research Program (SHRP) has shown that the large molecular size (LMS) fraction is composed predominantly of a collection (conglomerate) of smaller, highly

associated polar molecules which thus appear in the chromatogram as molecules of "apparent" large molecular size. Thus, this technique has the potential of characterizing the strongly associating molecular components that play a major role in determining the rheological properties and aging characteristics of asphalt cement related to pavement performance.

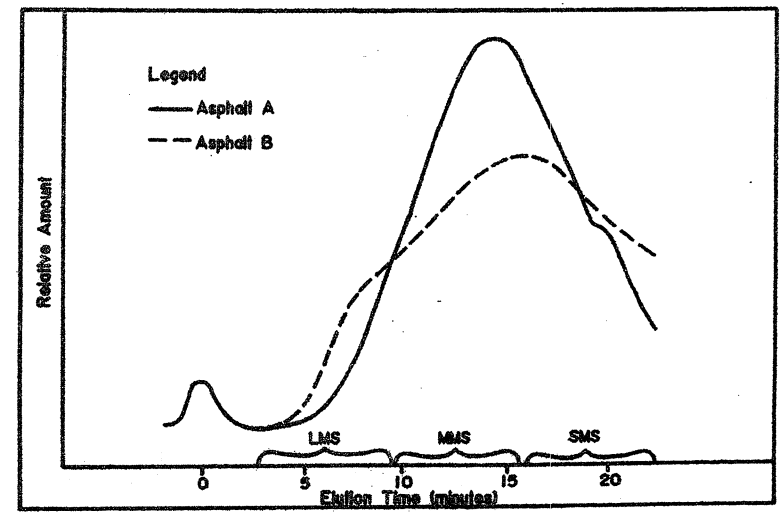


Figure 2–58. Typical Chromatograms of Whole Asphalts (after Jennings *80*)

SHRP Research in Asphalt Chemistry

Advances in analytical technology were used in SHRP to better understand the asphalt chemistry. Size exclusion chromatography (SEC) was used to isolate two fractions (called SEC I and SEC II) from the asphalts. The SEC method was modified by using toluene as the solvent rather than the strong (polar) solvent like tetrahydrofuran used by Jennings in earlier work (*80*). The weaker solvent toluene did not disrupt the asphalt structure as much as tetrahydrofuran. SEC I was believed to contain "associated molecules" and SEC II was believed to contain

"solvent molecules". Additional analysis demonstrated that the majority of the polar materials were present in SEC I, while the nonpolar materials were in the SEC II. Performance based asphalt physical tests developed in the SHRP indicated that increased amounts of SEC I generally caused brittleness, low temperature cracking, and fatigue cracking (thin HMA pavements). On the other hand, increased amounts of SEC II caused rutting and fatigue cracking (thick HMA pavements).

Ion Exchange Chromatography (IEC) was also used as a powerful analytical technique to understand molecular interactions in asphalts. In this method, the asphalt is dissolved in a solvent and then passed through a series of ion–exchange resins. Resin beds, basic and acidic in succession, are used to attract the polar molecules according to their acidic or basic character. The nonpolar molecules are not retained in the resin beds and are collected as a separate fraction. The IEC thus permitted to separate asphalt molecules based on the very characteristic (polarity) which contributes to the performance of asphalt.

According to SHRP researchers, asphalt is believed to be a 3–dimensional molecular matrix of highly polar molecules swollen by its more neutral molecules. The 3–dimensional nature makes it somewhat elastic, but, with sufficient stress (thermal or mechanical) the weak bonds rearrange and the asphalt experiences permanent strain.

The IEC analytical technique was also used to isolate a fraction termed "amphoterics". This fraction is believed to be the key building block of asphalts. The term amphoterics is used to describe an asphalt molecule which has both an acid and basic group in the molecule, but not at the same site. Although amphoterics constitute only 10 to 15 percent of asphalts, they are largely responsible for the viscoelastic properties of asphalts based on the test data obtained in SHRP.

Asphalt chemistry is very complex. Researchers have debated in the past and continue to debate on this very interesting subject. Reference *81* contains SHRP research data on asphalt chemistry.

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CHAPTER 3. AGGREGATES

INTRODUCTION

Aggregates used in highway construction are largely obtained from local supplies of natural rock. The natural rocks occur as either outcrops at or near the surface or as gravel deposits usually along old stream beds. A consequence of using locally found aggregate is that certain regions will have excellent quality aggregate and other regions will have marginal quality aggregate.

Natural rocks are classified by geologists into three groups depending on their origin—igneous, sedimentary, and metamorphic. Other types of aggregate that are sometimes used for HMA are lightweight aggregate, produced by heating clay or shale to very high temperatures, and slag, normally produced in the blast furnace during steel production. These two artificial aggregates have provided good skid resistance when used in HMA.

Igneous rocks are primarily crystalline and are formed by the cooling of molten rock magma as it moves toward or on the surface of the earth. Igneous rocks are classified based on size of the crystal grains and on composition as either acidic or basic. The crystal grains are classified as coarse if they are larger than 2 mm and fine when smaller than 0.2 mm. Classification of igneous rocks based on composition is shown below:

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	Acidic	Intermediate	Basic
Silica, %	> 66	55–66	< 55
Specific gravity	< 2.75	—	> 2.75
Color	Light	—	Dark
Presence of free quartz	Yes	—	No

Sedimentary rocks are primarily formed either by the deposition of insoluble residue from the disintegration of existing rocks or from deposition of the inorganic remains of marine animals. Classification is based on the predominant mineral present as calcareous (limestone, chalk, etc.), siliceous (chert, sandstone, etc.), or argillaceous (shale, etc.).

Metamorphic rocks are igneous or sedimentary rocks that have been subjected to heat and/or pressure sufficient to change their mineral structure so as to be different from the original rock. Metamorphic rocks are generally crystalline in nature with grain sizes from fine to coarse.

Gravel is formed from the breakdown of any natural rock. Gravel particles are found in existing or ancient waterways, and the particles are usually smooth and typically rounded or subrounded by wear as the material is moved along the waterway by the action of water. Gravel is usually required to be crushed prior to use in HMA.

Natural sands consist primarily of the most resistant final residue of the deterioration of natural rocks. The predominant mineral in sand is normally quartz. The grain size normally ranges from a No. 8 (2.36mm) sieve down to dust size particles (No. 200 or 75micron sieve). Many sand deposits often contain silt and/or clay particles in various quantities and may be required to be washed prior to use in HMA. Many northern states have a great deal of glacial sands and gravels which have less clay than most river gravels and sands.

Slag is a byproduct of metallurgical processing and is typically produced from processing of steel, tin, and copper. Slags typically have

the properties of igneous rocks and their structure can vary from glassy to honeycombed. Blast furnace slag produced during the processing of steel is the most widely used of the slags for pavement construction applications. This aggregate produces a high quality asphalt mix with good skid resistance. The absorption of slag is often high; hence, the amount of asphalt binder required when slag aggregates are used is usually higher than that for naturally occurring aggregates.

For highway construction purposes it is not usually necessary to have extensive geologic information about the rocks. The important characteristics of the rocks relate to how well the materials serve in the various applications such as subbases, bases, or in the various HMA courses used in pavement construction. Therefore, for the most part, it is the physical properties of the rocks which are important for highway construction. The chemical properties, which are not well understood, are important to ensure good bond between the asphalt and aggregate. Since the natural rock materials coming from different areas within a quarry or gravel pit can vary, it is important to sample and test the material on a regular basis to ensure that aggregate properties are consistent and meet the specified standards.

AGGREGATE PRODUCTION

An aggregate's physical characteristics, such as resistance to abrasion and strength, are determined primarily by the characteristics of the parent rock. However, the production process in the quarry can significantly improve the quality of the aggregate by the elimination of weaker rock layers and by the effect of crushing on the particle shape and gradation of the aggregate.

Almost all quarries have a layer of overburden material that must be removed before sound rock is encountered. The overburden typically consists of soil and decomposed rock. The demarcation line between sound rock and decomposed rock is often difficult to establish. In addition, decomposition often occurs along cracks in the rock or at joints

between rock layers. Therefore it is important to eliminate any inferior rock by stripping and wasting the soil overburden and decomposed rock. Scalping material at the primary crusher allows the pit fines to bypass the crusher (by passing the aggregate over a grizzley), and the rock to be further processed into various sizes downstream through the finishing plant.

To provide an overall idea of a crushing operation, a schematic diagram and brief description of a simplified crusher set–up are provided in the following paragraphs.

Quarry Operations

The basic purpose of a quarry operation is to remove sound rock from the face of the quarry by blasting or other method and then using a series of crushers, pulverizers, and screening units to size the materials into a sufficient number of components to produce the desired pavement construction materials. It is also desirable to produce crushed aggregate that is essentially cube–shaped and not flat or elongated. A simplified layout for a crushing operation is shown in Figure 3–1. Notice that a jaw crusher is used as the primary crusher, and a standard cone crusher is used as the secondary crusher. These selected crushing units are chosen to:

1. Produce the desired product size and shape;
2. Reduce the largest size of rock normally received;
3. Have sufficient capacity to handle the peak loads;
4. Minimize choking and plugging;
5. Safely handle uncrushable material, like iron, by yielding instead of breaking;
6. Require a minimum of personnel;
7. Satisfy crushing requirements without the need for additional crushing stages and auxiliary equipment;
8. Provide lowest power demand per ton of finished product;
9. Resist abrasive wear;

10. Operate economically with a minimum of maintenance; and
11. Offer long, dependable service life.

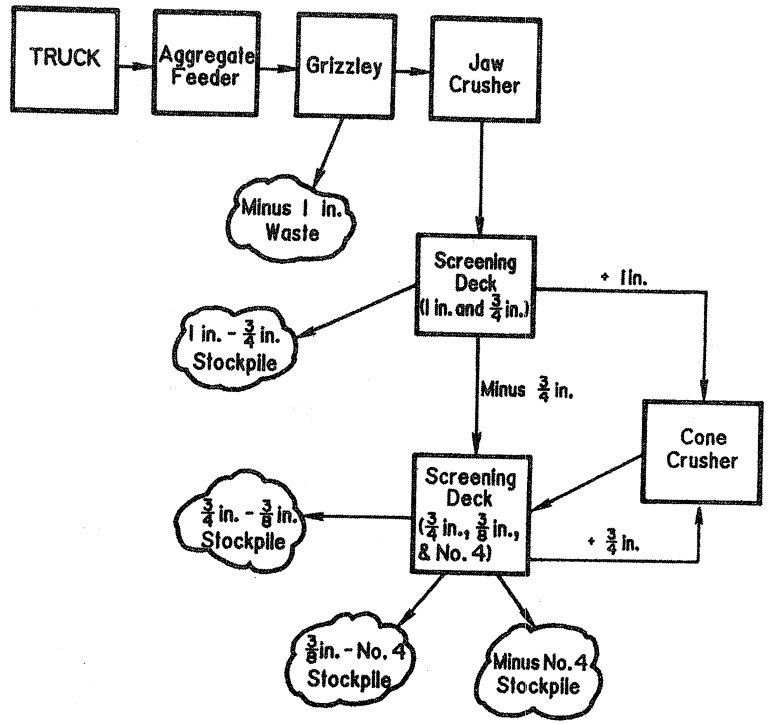


Figure 3–1. Simplified Crusher Set–up

The overburden is removed and wasted prior to blasting the quarry face. Blasting is a dangerous operation and must be performed by a qualified individual who is licensed to perform this type work. The rock, removed from the quarry floor after blasting, is transported to the crusher by truck. After blasting, the weathered rock around cracks and at other locations typically breaks into smaller pieces than the remaining sound rock. These smaller pieces are removed with a grizzley prior to being crushed. For the example shown in Figure 3–1, all material smaller than one inch is removed and wasted. The material larger than one inch that passes over the grizzley is fed into a jaw crusher (primary crusher) that

breaks the rock down to a size that can be handled by other crushers. Suppose for this simplified crushing operation that four aggregate sizes will be produced: 1 inch–3/4 inch (25mm–19mm), 3/4 inch–3/8 inch (19mm–9.5mm), 3/8 inch–No. 4 (9.5mm–4.75mm), and minus No. 4 (4.75mm). The next operation after the jaw crusher is to screen the aggregate into various sizes. The first screen deck separates aggregate into three sizes by using a l–inch (25mm) and a 3/4–inch (19mm) screen. The material larger than 1 inch (25mm) is fed to the cone crusher for additional crushing. The material smaller than 1 (25mm) inch but larger than 3/4 inch (19mm) is stockpiled. The material smaller than 3/4 inch (19mm) is fed to a second screen deck for further screening. The second screen deck uses three screen sizes 3/4 inch (19mm), 3/8 inch (9.5mm), and No. 4 (4.75mm). The material larger than 3/4 inch (19mm) is fed back to the cone crusher for further crushing. The other three material sizes are stockpiled as shown. This is a very simplified crushing operation that identifies methods that are normally used to crush the aggregate and separate it into sizes. Most operations are much more complicated than this but, regardless of the complexity, the operation is almost entirely made up of crushers and screening decks. Many operations also have the capability to wash the aggregate as needed at certain points in the operation.

During the operation of a crushing facility, it is absolutely essential that the properties of the final product be consistent. This requires that the parent rock being fed into the crusher has consistent physical properties such as specific gravity and absorption. When rock is removed from various locations in the quarry, the physical properties of the aggregate may vary substantially, thus producing problems in maintaining consistent air voids in the HMA. Quality control during the crushing operation must ensure that physical properties of the aggregate do not vary excessively. Ideally, the amount of material being fed to the crushing operation should be approximately constant. If the rate of material feed is increased or decreased, the amount of overflow on particular screen sizes changes, especially when operating near full capacity for individual screens. For instance, increasing the rate of flow

of material over a screening deck may result in more overflow of finer material over an individual screen. This increase in percentage of overflow results in finer material being fed into the aggregate stockpile. The feed rate must exceed some minimum amount before the efficiency of the screen drops significantly below 100% resulting in a measurable change in gradation.

The primary crusher provides the initial mechanical reduction of the pit run material down to maximum sizes ranging from 8 inches to 1 inch. Secondary and tertiary crushing refers to the additional stages of crushing that are required to reduce the aggregate to some desired size. For natural materials from old river beds and streams, such as gravels, the primary jaw crusher is usually not needed since the material is typically small enough that primary crushing is not required.

Mechanical reduction involves four different mechanisms: impact, attrition, shearing and compression. Most crushers utilize a combination of all four mechanisms. A brief description of each mechanism is provided in the following paragraphs.

Impact refers to the sharp, instantaneous impingement of one object against another. In crushing operations, impact may be created by way of gravity or dynamic impact (Figure 3–2). The type of crusher (Figure 3-3) used depends on the type of aggregate. Harder rock results in the use of more compression crushers than impact.

Attrition refers to the reduction of materials by a rubbing action between two hardened surfaces and is primarily a grinding method (Figure 3–4). Hammermills, gyratories, and cones use this type of reduction method.

Shear crushing occurs as a result of a trimming or cleaving action such as that produced by the single roll crusher shown in Figure 3–5. Shear crushing is usually combined with other crushing mechanisms such as impact and compression.

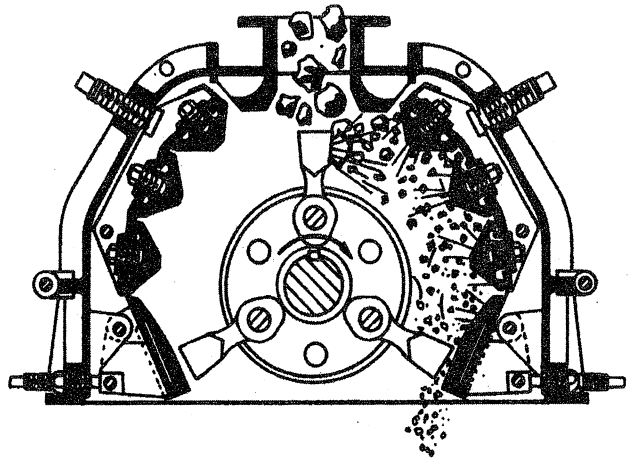


Figure 3–2. Mechanical Reduction by Impact (*1*)

CRUSHERS	REDUCTION METHODS			
	Impact	Attrition	Shearing	Compression
IMPACTORS	●			
VERTICAL IMPACT PULVERIZERS	●			
BRADFORD BREAKERS	●			
HAMMERMILLS	●	●	●	
RING-TYPE GRANULATORS	●		●	●
SINGLE-ROLL CRUSHERS	●		●	●
GYRATORIES	●	●		●
JAW CRUSHERS	●			●
CONE		●	●	●

Figure 3–3. Display of the Reduction Methods Employed by Different Types of Crushers (*1*)

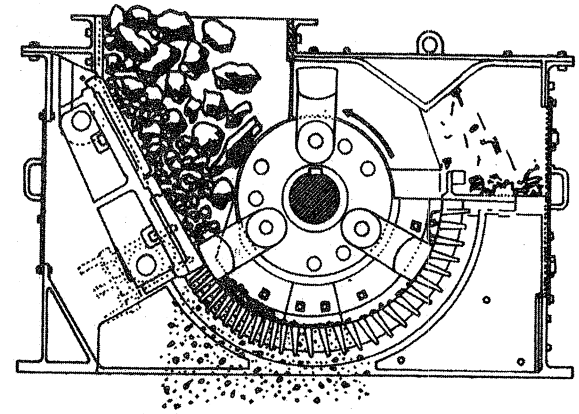


Figure 3–4. Mechanical Reduction by a Combination of Impact and Attrition in a Hammermill (*1*)

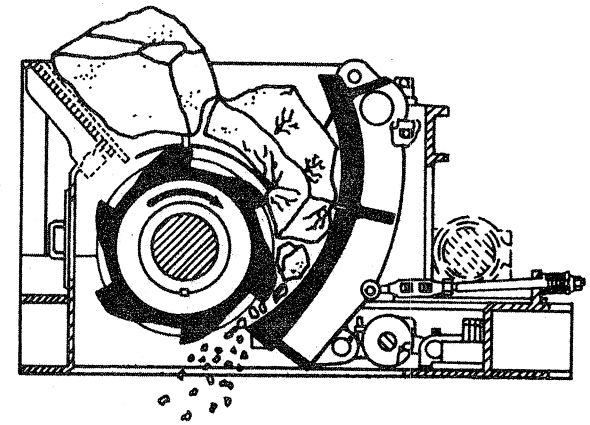


Figure 3–5. Mechanical Reduction by a Combination of Shear, Impact, and Compression in a Single–roll Crusher (*1*)

Compression refers to the mechanical reduction by compressive forces between two surfaces (Figure 3–6). Jaw crushers using compression are the most effective system for reducing the size of larger particles of very hard and abrasive rocks.

The types of crushers for a particular application must be carefully selected to produce a material meeting the customer's specifications

while operating at minimum cost. Obviously the type of rock being processed has a significant influence on the choice of crushing equipment to be used.

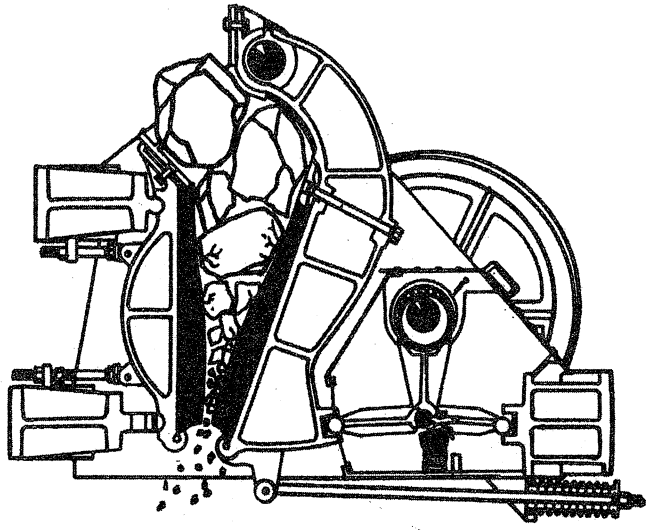


Figure 3–6. Mechanical Reduction by Compression and Impact in a Specially Designed Jaw Crusher that Crushes Without Rubbing (*1*)

AGGREGATE SAMPLING

Prior to conducting any tests on aggregates, samples must be obtained from some source using proper sampling techniques (ASTM D75). The samples may be randomly selected or may be selected to be representative (average of the material being sampled), depending on the purpose of samples. Usually for mix design, representative samples are taken, and for quality control, random samples are taken. When there is a need to characterize the average properties of a stockpile or other source of material, representative samples are required. However, during the construction process, random samples are normally taken so that a statistical analysis can be made concerning the overall quality of mixture being produced. If proper types of samples are not taken, all tests conducted on the aggregate may be difficult or impossible to analyze,

and this may result in the design or construction of an HMA mixture that has poor quality, resulting in overall unsatisfactory performance.

Samples of aggregate are normally taken from a stockpile, belt, hot bins, or sometimes from a loaded truck. The person taking samples must realize that the material segregates when stockpiled, loaded into trucks, or loaded into bins. Hence, the best location to take a sample is from a conveyor belt; however, the full width of flow must be sampled since the aggregate also segregates on the belt. In this case a random sample is taken by selecting a random time to take the sample from the belt. A representative sample can be obtained by taking and combining a number of random samples throughout some desired time period, such as one day for belt samples, or by taking samples from various locations in a stockpile and combining these samples.

One should always use safe practices when working around stockpiles, trucks, etc. to minimize the possibility of injury. For example samples from stockpiles are normally taken on the side of the piles. It can be unsafe climbing up a stockpile since a large amount of aggregate can quickly slide down the side of the pile injuring anyone in the area.

Representative samples are almost always used to evaluate stockpiles. Again, samples must be taken in a way to minimize the effect of stockpile segregation. The aggregate at the bottom edge of a stockpile is usually coarser than the rest of the stockpile. Hence, samples should be taken at several locations away from the bottom and top of the stockpiles and combined to form a representative sample. For production control it is better to take belt samples than to take samples from stockpiles.

After taking the required number and amount of samples and bringing them back to the laboratory, each sample must be reduced to the appropriate size for testing. Samples can be reduced with a sample splitter or by quartering (ASTM C702). Both methods work successfully, but one of these must be utilized if an accurate sample is to be obtained for tests. The importance of sampling and splitting to test size cannot be overemphasized since all test results are dependent on the use of proper techniques.

MINERALOGY AND CHEMICAL PROPERTIES

As discussed previously, it is primarily the physical properties of aggregates (to a lesser extent, the chemical properties) that determine the suitability for use in HMA. Basic physical/mechanical properties (such as density, porosity, and strength) and chemical/physicochemical properties (such as wetting, adhesion, and stripping) are functions of the composition and structure of the minerals in the aggregate. An understanding of the mineralogy and identification of minerals can provide information on the potential physical and chemical properties of an aggregate for a given use, and can help avoid the use of aggregate containing undesirable mineral constituents. The project specification requirements should be prepared to ensure that aggregates having undesirable mineral components are not accepted for use.

Aggregate Mineralogy

Common Minerals Found in Aggregates. A majority of natural aggregates are composed of a combination of minerals. Among the most important minerals found in aggregates are silica minerals (quartz); feldspars (orthoclase, plagioclase); ferromagnesian minerals (muscovite, vermiculite); carbonate minerals (calcite, dolomite); and clay minerals (illites, kaolinites, montmorillonites). These minerals and associated rocks are described in ASTM C294 (Table 3–1) (*2*). Table 3–2 gives the average mineral compositions commonly found in aggregates (*3*).

Table 3–1. Rock and Mineral Constituents in Aggregates (*2*)

Minerals	Igneous rocks	Metamorphic rocks
Silica	Granite	Marble
Quartz	Syenite	Metaquartzite
Opal	Diorite	Slate
Chalcedony	Gabbro	Phyllite
Tridymite	Peridotite	Schist
Cristobalite	Pegmatite	Amphibolite
Silicates	Volcanic glass	Hornfels
Feldspars	Obsidian	Gneiss
Ferromagnesian	Pumice	Serpentinite
Hornblende	Tuff	
Augite	Scoria	
Clay	Perlite	
Illites	Pitchstone	
Kaolins	Felsite	
Chlorites	Basalt	
Montmorillonites		
Mica	**Sedimentary rocks**	
Zeolite		
Carbonate	Conglomerate	
Calcite	Sandstone	
Dolomite	Quartzite	
Sulfate	Graywacke	
Gypsum	Subgraywacke	
Anhydrite	Arkose	
Iron sulfide	Claystone, siltstone,	
Pyrite	argillite, and shale	
Marcasite	Carbonates	
Pyrrhotite	Limestone	
Iron oxide	Dolomite	
Magnetite	Marl	
Hematite	Chalk	
Goethite	Chert	
Ilmenite		
Limonite		

For brief descriptions, see "Standard Descriptive Nomenclature of Constituents of Natural Mineral Aggregates" (ASTM C294).

Table 3–2. Average Mineral Composition for Rocks (3)

Rock	Quartz	Orthoclase	Plagioclase	Augite	Hornblende	Biotite	Muscovite	Epidote	Rock Glass
Igneous Rocks									
Granite	30	41	8	-	-	3	3	1	-
Diorite	8	7	30	3	27	4	0.1	5	-
Gabbro	0.5	-	44	28	9	2	-	1	-
Diabase	-	-	44	46	-	-	-	-	2
Rhyolite	32	45	3	-	0.7	3	2	2	0.4
Trachyte	3	42	1	2	6	0.5	-	8	0
Andesite	0.6	-	48	14	3	-	-	3	13
Basalt	-	-	36	35	-	-	-	-	21
Metamorphic Rocks									
Quartzite	84	3	-	-	1	2	2	2	-
Feldspathic-quartzite	46	27	1	-	-	2	5	1	-
Hornblende-gneiss	10	16	15	3	45	3	1	?	-
Granite-gneiss	37	32	3	-	-	7	11	2	-
Biotite-schist	34	13	3	-	1	38	3	2	-
Mica-schist	37	16	1	-	-	13	26	2	-
Slate	29	4	-	-	-	-	55	2	-
Marble	3	0.2	0.2	-	-	-	Calcite = 96		-
Amphibolite	3	1	8	-	70	1	[illegible]	12	-
Sedimentary Rocks									
Sandstone	79	5	0.3	-	-	0.2	1	-	-
Feldspathic-sandstone	35	26	2	-	-	0.6	2	1	-
Calcareous-sandstone	46	3	2	-	-	-	Calcite = 42	-	-
Chert	93	-	-	-	-	-	Calcite = 1	-	-
Limestone	6	-	-	-	Dolomite = 8		Calcite = 83	-	-
Dolomite	5	-	-	-	Dolomite = 82		Calcite = 11	-	-

Effect of Mineralogy on Performance. Since minerals have definite chemical composition and usually specific crystalline structure, the physical and chemical properties of aggregates can be expected to be

associated with mineralogy of the aggregates. Table 3–3 gives typical engineering properties for common aggregate types, and Table 3-4 summarizes desirable properties of rocks for HMA aggregates. Most aggregates are composed of several minerals, often with variable compositions. Even with aggregates of uniform mineralogy, the properties may be altered by oxidation, hydration, leaching, weathering and foreign coatings. Therefore, mineralogy alone cannot provide a basis for predicting the behavior of an aggregate in service. Petrographic examinations are useful, and past performance of similar aggregates under similar environmental and loading conditions can be helpful in evaluating aggregates.

Table 3–3. Summary of Engineering Properties of Rocks (4)

Type of Rock	Mechanical Strength	Durability	Chemical Stability	Surface Characteristics	Presence of Undesirable Impurities
Igneous:					
Granite, syenite, diorite	Good	Good	Good	Good	Possible
Felsite	Good	Good	Questionable	Fair	Possible
Basalt, diabase, gabbro	Good	Good	Good	Good	Seldom
Periodotite	Good	Fair	Questionable	Good	Possible
Sedimentary:					
Limestone,dolomite	Good	Fair	Good	Good	Possible
Sandstone	Fair	Fair	Good	Good	Seldom
Chert	Good	Poor	Poor	Fair	Likely
Conglomerate, breccia	Fair	Fair	Good	Good	Seldom
Shale	Poor	Poor	Poor	Good	Possible
Metamorphic:					
Gneiss, schist	Good	Good	Good	Good	Seldom
Quartzite	Good	Good	Good	Good	Seldom
Marble	Fair	Good	Good	Good	Possible
Serpentinite	Fair	Fair	Good	Fair to poor	Possible
Amphibolite	Good	Good	Good	Good	Seldom
Slate	Good	Good	Good	Poor	Seldom

Table 3–4. Desirable Properties of Rocks for HMA (5)

Rock Type	Hardness, Toughness	Resistance to Stripping†	Surface Texture	Crushed Shape
Igneous				
Granite	Fair	Fair	Fair	Fair
Syenite	Good	Fair	Fair	Fair
Diorite	Good	Fair	Fair	Good
Basalt (trap rock)	Good	Good	Good	Good
Diabase (trap rock)	Good	Good	Good	Good
Gabbro (trap rock)	Good	Good	Good	Good
Sedimentary				
Limestone, dolomite	Poor	Good	Good	Fair
Sandstone	Fair	Good	Good	Good
Chert	Good	Fair	Poor	Good
Shale	Poor	Poor	Fair	Fair
Metamorphic				
Gneiss	Fair	Fair	Good	Good
Schist	Fair	Fair	Good	Fair
Slate	Good	Fair	Fair	Fair
Quartzite	Good	Fair	Good	Good
Marble	Poor	Good	Fair	Fair
Serpentine	Good	Fair	Fair	Fair

†Aggregates that are hydrophilic (water-loving) tend to strip more readily since water more easily replaces an asphalt film. Freshly crushed aggregates with many broken ionic bonds also tend to strip more readily.

In addition to the properties listed in Table 3–3 and Table 3–4, mineral composition of coarse aggregates also affects the skid resistance of HMA. Quartz and feldspar are harder and more polish resistant minerals which are normally found in igneous rocks such as basalt and gabbro. On the other hand, calcite which occurs in limestones, is an example of a soft mineral. Limestones that have a high percentage of soft materials tend to polish more rapidly than most other aggregate types. The acid insoluble residue test (ASTM D3042) has been used to measure the amount of harder materials present in carbonate aggregates. Many

highway agencies specify a minimum of 10 percent acid insolubles to assure acceptable frictional properties. Other agencies use a polishing test (ASTM D3319 or E660 and E303) or petrographic examination (ASTM C295) to evaluate the polishing potential of an aggregate.

Another factor which affects an aggregate's usefulness in HMA, somewhat related to mineralogy, is the presence of surface coatings and other deleterious substances (4). These deleterious substances may include clay, shale, silt, iron oxides, mica, gypsum, water–soluble salts, and other soft particles that affect proper bonding with asphalt. Deleterious materials may also increase the moisture susceptibility of an asphalt mixture. Aggregates with deleterious substances are undesirable for HMA and should not be used unless the amount of foreign matter is reduced by washing or other means. Specifications for aggregates normally list the deleterious materials (e.g., clay lumps, shale, soft particles) and limit their quantity (e.g., 3 to 6 percent). Other agencies specify a minimum sand equivalent requirement (ASTM D2419) or a maximum plasticity index (ASTM D4318) which is discussed later. AASHTO M283 suggests that coating on aggregates after dry sieving should not exceed 0.5 percent when tested by AASHTO T11 (ASTM C117).

One of the most important effects of aggregate mineralogy on the performance of HMA is its influence on adhesion and moisture damage. Asphalt cement bonds better to aggregates with certain mineral types. For instance, asphalt cement normally bonds better to carbonate aggregates (limestone) than to siliceous aggregates (gravel).

Petrographic Examination. A petrographic examination of an aggregate is a visual examination of individual aggregate particles using an optical microscope, sometimes supplemented by X–ray diffraction, differential thermal analysis, electron microscope, or chemical analysis. The petrographic examination can be used to measure the relative abundance of rock and mineral types, the physical and chemical attributes (such as particle shape, surface texture, hardness, pore characteristics, and chemical activity), and the presence of harmful

contaminants. ASTM C295 describes the detailed procedure for a petrographic examination of aggregates. This test has been used for many years to evaluate aggregates for use in concrete but has not been widely used for aggregates for HMA. The mineralogy of an aggregate has an aggregate has a greater effect on performance of concrete mixes than it does for asphalt mixes.

Chemical Properties of Aggregate

Chemical properties of an aggregate identify the chemical composition and/or indicate the transformation that an aggregate can undergo due to chemical action. Chemical compositions of aggregates based on chemical analysis are usually given in terms of oxides, regardless of whether such oxides are actually present in the sample. These data are not as informative as mineral composition and sometimes may be confusing and misleading. Some aggregates contain substances which (1) are soluble in water (e.g., gypsum); (2) are subject to oxidation, hydration, and carbonation; and (3) can react with compounds in Portland cement, although reactivity of these aggregates with asphalt cement has not been conclusively established. Chemical properties of aggregates have little effect on their suitability and performance, except as they affect adhesion of the asphalt binder to the aggregate and compatibility with antistripping additives that may be incorporated into the asphalt binder.

Asphalt cement must wet the aggregate surface, stick to the aggregate, and resist stripping of the asphalt film in the presence of water. Therefore, the surface chemistry of the aggregate particles plays an important role in performance. The adhesion of asphalt cement to aggregate and the displacement of the asphalt film by water (stripping) is a complex phenomenon involving the physicochemical interaction between many parameters. Several theories have been suggested on the mechanisms of adhesion and stripping (*4*). It appears thus far that none can fully explain the phenomenon and that more than one mechanism may be operating at the same time. However, it has been firmly

established that mineralogy and chemical composition of the aggregate are of primary importance in the stripping phenomenon.

There has been abundant evidence to indicate that some aggregates appear to have a greater affinity for water than for asphalt cement, and asphalt films on these aggregate particles may become detached or stripped after exposure to water (*6*). These aggregates are called hydrophilic (water–loving), and they tend to be acidic in nature. On the other hand, aggregates having an affinity for asphalt cement are called hydrophobic (water–hating), and they tend to be basic in nature. It is commonly accepted that the nature of electric charges on the aggregate surface when in contact with water significantly affects the adhesion between the aggregate and the asphalt cement and its resistance to moisture damage. Most siliceous aggregates such as sandstone, quartz and siliceous gravel become negatively charged in the presence of water, while limestone and other calcareous materials bear a positive charge in the presence of water. Many aggregates contain both types of charges because they are composed of minerals such as silica with negative charges, and also calcium, magnesium, aluminum, or iron with positive charges. Typical aggregates that bear mixed charges include traprock, basalts, porphyries, and the siliceous limestones. Dolomite and quartz are examples of extreme cases of electropositive and electronegative aggregates, respectively. A classification of aggregates lying between these extremes is shown in Figure 3–7 (*7*).

Since stripping is one of the major distresses affecting HMA performance, the effect of aggregates on the moisture sensitivity of HMA must be evaluated. A number of practical tests are available to predict the compatibility of aggregate and asphalt cement. These tests are discussed in Chapter 5.

There has been evidence to suggest that asphalt–aggregate interactions may cause asphalt molecular structuring at the aggregate surface which affects the bonding and setting properties of HMA (*8*). However, the precise mechanisms and the role of chemical/mineralogical composition of the aggregate on the properties of asphalt cements and asphalt mixes are not yet well understood.

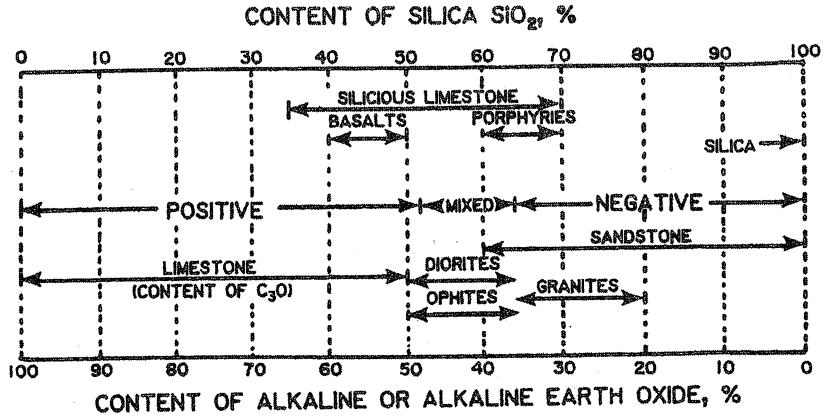


Figure 3–7. Classification of Aggregates by Surface Charges (7)

PHYSICAL PROPERTIES OF AGGREGATES

Aggregates for HMA are usually classified by size as coarse aggregates, fine aggregates, or mineral fillers. ASTM defines coarse aggregate as particles retained on a No. 4 (4.75mm) sieve, fine aggregate as that passing a No. 4 sieve (4.75mm), and mineral filler as material with at least 70 percent passing the No. 200 (75microns) sieve. Other agencies may use the No. 8 (2.36 mm) sieve (e.g., the Asphalt Institute) or the No. 10 (2.00 mm) sieve as the dividing line between coarse and fine aggregates.

Aggregates for HMA are generally required to be hard, tough, strong, durable (sound), properly graded; to consist of cubical particles with low porosity; and to have clean, rough, hydrophobic surfaces. Table 3–5 summarizes the various properties that an aggregate must possess in order to fulfill its function and the relative importance of aggregate in HMA (9).

Specifications for coarse aggregates, fine aggregates, and mineral fillers are given in ASTM D692, D1073 and D242, respectively. The suitability of aggregates for use in HMA is determined by evaluating the following material characteristics:

Table 3–5. Summary of Aggregate Properties to Meet Function of System (9)

Function	Aggregate Property	Relative Importance[1] of Property in Hot Mix Asphalt
Have adequate internal strength and stability to distribute surface pressures and to prevent extensive surface deflections	1. Mass stability	I
	2. Particle strength	I
	3. Particle stiffness	I
	4. Particle surface texture	I
	5. Particle shape	I
	6. Grading	I
	7. Maximum particle size	I
Resist deteriorating effects of weather and chemicals	1. Resistance to attack by chemicals	U
	2. Solubility	U
	3. Slaking	I
	4. Resistance to wetting-drying	U
	5. Resistance to freezing-thawing	U
	6. Pore structure	I
Resist deteriorating effects of applied loads	1. Resistance to degradation	I
Resist the effects of internal forces, such as expansion, contraction and warping	1. Volume change-thermal	N
	2. Volume change-wetting and drying	N
	3. Pore structure	
	4. Thermal conductivity	N
		N
Be compatible with binder used in system	1. Chemical compounds reactivity	I
	2. Organic material reactivity	N
	3. Coatings	I
	4. Volume stability	N
	5. Base exchange	I
	6. Surface charges	I
	7. Pore structure	N

Table 3–5. Summary of Aggregate Properties to Meet Function of System (Cont.) (*9*)

Function	Aggregate Property	Relative Importance[1] of Property in Hot Mix Asphalt
Retain acceptable standards of performance at the surface by:		
a. Maintaining adequate skid resistance	1. Particle shape	I
	2. Particle surface texture	I
	3. Maximum particle size	I
	4. Particle strength	I
	5. Wear resistance	I
	6. Particle shape of abraded fragments	I
	7. Pore structure	I
b. Having acceptable surface roughness characteristics	1. Maximum particle size	I
	2. Grading	I
c. Minimizing glare and light reflectivity	1. Reflectivity	I
	2. Glare	I
d. Preventing the occurrence of loose material	1. Resistance to degradation	I
	2. Specific gravity	I
e. Minimizing tire wear	1. Particle shape	I
	2. Particle surface texture	I
	3. Maximum particle size	I
f. Minimizing rolling resistance	1. Maximum particle size	I
	2. Particle shape	I
g. Minimizing noise level	1. Maximum particle size	I
	2. Particle shape	I
h. Preventing electrostatic buildup	1. Electrical conductivity	I
Retaining properties during the construction process that support all other functions of the system	1. Maximum particle size	I
	2. Resistance to degradation	I
	3. Integrity during heating	I

[1]Notes: I - Important; N - Not Important; U - Importance Unknown

1. Size and gradation;
2. Cleanliness/deleterious materials;
3. Toughness/hardness;
4. Durability/soundness;
5. Surface texture;
6. Particle shape;
7. Absorption; and
8. Affinity for asphalt.

Affinity for asphalt is related to surface chemistry and was discussed earlier. Other physical properties and the tests commonly used for evaluation are discussed in the following sections. In addition, the various types of specific gravity of aggregates are discussed since these are used to calculate voids in mix design. The most important characteristics of aggregates for HMA, their significance, commonly used tests to evaluate these properties, and recommended test requirements are listed in Table 3–6.

Toughness and Abrasion Resistance

Aggregates, through internal friction, must transmit the wheel loads to the underlying layers and also be resistant to abrasion and polishing due to traffic. Aggregates are subject to crushing and abrasive wear during manufacturing, placing, and compaction of HMA. They must be hard and tough to resist crushing, degradation, and disintegration when stockpiled, fed through a HMA facility, placed with paver, compacted with rollers, and traveled over with trucks. The Los Angeles (L.A.) abrasion test and the micro-deval test are most often used to obtain an indication of the desired toughness and abrasion characteristics. Details of these two test methods are described in a later section.

Table 3–6. Characteristics and Tests of Aggregates for HMA

Characteristics	Significance	Test	Requirement
Hardness/toughness	Resistance to abrasion and degradation; quality of particles	ASTM C131 ASTM C535	Maximum percent weight loss; ASTM D692; D1073; AASHTO M283
Soundness	Resistance to freeze and thaw; wet and dry	ASTM C88 AASHTO T103	Maximum percent weight loss; ASTM D692; D1073; AASHTO M283
Particle shape and surface texture	Friction; skid-resistance; compaction; mix stability	ASTM D3398 ASTM D4791	Minimum percent of crushed particles; ASTM D692; AASHTO M283
Resistance to polishing	Skid and wear resistance	ASTM D3319 ASTM E303 ASTM E660 ASTM D3042	Maximum polish value; Minimum acid insolubles
Durability	Resistance to effect of weather and aging	ASTM D3744	Maximum durability index
Resistance to stripping	Resistance to effect of water	ASTM D1664 NCHRP 246; 274 AASHTO T283 ASTM D1075 AASHTO T182	Minimum percent of coated area; Percent retained strength or modulus; Number freeze-thaw cycles to cause crack; AASHTO M283
Specific gravity and water absorption	Mix design calculations; porosity	ASTM C127 ASTM C128	---
Asphalt absorption	Mix design calculations; durability	ASTM D2041 ASTM D4469	---
Gradation and size	Mix stiffness, stability, workability, fatigue resistance, fractural strength, durability, permeability, voids, compactibility	ASTM C117 ASTM C136 & C702 AASHTO T30	Minimum and maximum percentages passing standard sieves; ASTM D3515
Mineral composition	Wettability and stripping potential; porosity	ASTM C294 ASTM C295	---
Cleanliness and deleterious substances	Resistance to weathering and effects of water	ASTM C117 & D422 ASTM C123 ASTM C142 ASTM D2419 ASTM D4318	Maximum clay lumps and soft particles of one percent; Minimum sand equivalent of 25-45%; P.I. not exceeding 4-6; Percent fines (ASTM D242; D1073); AASHTO M283

Methods that are available include ASTM C131 for coarse aggregate smaller than 1.5 inch (38mm) and ASTM C535 for testing aggregates larger than 0.75 inch (19mm) and up to 3 inch (76mm) maximum size. ASTM C131 is performed by mixing 5,000 g of aggregate blended to meet one of several gradings, along with a charge of six to twelve steel balls, each approximately 2 inches (51mm) in diameter and approximately 400 g in weight. This mix is tumbled inside a steel drum for 500 revolutions at a speed of 30–33 rpm. This is the method that is normally used for HMA aggregates. ASTM C535 is performed with 10,000 g of blended aggregate, twelve steel balls, and 1,000 revolutions. After tumbling, the fines that pass a No. 12 (1.68mm) sieve are weighed, and the percentage loss by weight of original sample is calculated as the Los Angeles abrasion. Initially, none of the material would pass a No. 12 sieve. Hence if 40 percent of the material passed a No. 12 sieve after the test, the L.A. abrasion would be 40.

Typical test values range from 10 percent for extremely hard igneous rocks to 60 percent for soft limestones and sandstones. Obviously, a number of aggregates will exceed this typical range. The maximum wear for coarse aggregate used in high–type HMA is typically limited by specifications to 40 percent by some agencies to as much as 60 percent by other agencies. ASTM D1073 does not specify a maximum abrasion loss for fine aggregates for any purpose.

The Los Angeles abrasion test is primarily a measure of the resistance of coarse aggregate to degradation by abrasion and impact; however, field observations do not show a good relationship between L.A. abrasion loss and performance. This test does not appear to be fully satisfactory for use with slags, cinders, and other lightweight aggregates. Experience has shown that many aggregates such as slag and soft limestones provide excellent performance even though the Los Angeles abrasion is high. One item that usually must be dealt with when producing HMA with aggregate having a high L.A. abrasion is the generation of dust during handling of aggregates and production of HMA. A high dust content will likely cause environmental problems as well as result in mixture control problems. The dust will tend to fill the

voids in an asphalt mixture resulting in lack of sufficient asphalt content to provide for good mixture durability.

The Micro-Deval test is used by many agencies to specify the resistance to wear for coarse aggregate. This test is similar to the Los Angeles abrasion test but it is conducted under water. Some believe that the L. A. abrasion test is more of an impact test and the Micro-Deval is a better abrasion test. The Micro-Deval test is conducted in accordance with ASTM D6928.

Durability and Soundness

Aggregates must be resistant to breakdown or disintegration under action of wetting and drying and/or freezing and thawing (weathering). The soundness test (ASTM C88) is an empirical screening test that is intended to provide an indication of durability due to weathering and is useful for evaluating new sources of aggregate for which no service records are available. This test involves submerging the aggregate in a solution of sodium or magnesium sulfate. During this test salt crystals grow in the aggregate pores and cause particles to disintegrate in some aggregates. Presumably this crystal formation simulates that of ice crystals and hence to some extent simulates the effect of freezing and thawing. An oven–dried sample of the aggregate is separated into specified sizes and immersed in a saturated solution of sodium or magnesium sulfate. The aggregate remains in the solution at a constant temperature for approximately 18 hours. The sample is then removed from the solution, dried to constant weight at 230 ± 9°F (110±5°C) and cooled. This cycle is typically repeated five times, after which the sample is washed to remove the salt and is dried. The loss in weight for each size fraction is determined by sieving, and the weighted average percent loss for the entire sample is computed (see ASTM C88 for sample calculation).

A number of tests to evaluate the effects of freezing and thawing have been proposed. Typically the procedures are similar to that of ASTM C88, except that different sized samples are immersed in water or 0.5 percent water–alcohol solution and then alternately subjected to

freezing and thawing for the required number of cycles (16 to 50). AASHTO T103 is one such procedure.

When specifications for aggregates contain a requirement for soundness by the sodium or magnesium sulfate test, the specifications generally state both the number of cycles and the maximum allowable percentage of loss. For example, ASTM D692 specifies a maximum of 12 percent loss after 5 cycles when using sodium sulfate and 18 percent loss when using magnesium sulfate since the action of the magnesium sulfate is more severe. The sulfate soundness test has been an accepted method of testing for many years. Despite this acceptance, it has been widely criticized, and numerous reports have appeared which describe its inability to accurately predict field performance for specific aggregates. Many engineers believe that freezing and thawing cycles do not create problems with aggregates in HMA. The aggregate is initially dried during production and hence should have little or no moisture in the pores immediately after production. (Aggregates with high absorption values probably do have some free moisture in the pores.) During HMA production, the aggregates are coated with a film of asphalt binder which should prevent the aggregate from absorbing a significant amount of moisture during the life of the mixture. Since there is little or no moisture in the aggregate, freezing and thawing should not be a significant problem.

Particle Shape and Surface Texture

Aggregate particles suitable for use in HMA should be cubical rather than flat, thin or elongated. In compacted mixtures, angular–shaped particles exhibit greater interlock and internal friction, and hence result in greater mechanical stability than do rounded particles. On the other hand, mixtures containing rounded particles, such as most natural gravels and sands, have better workability and require less compactive effort to obtain the required density. This ease of compaction is not necessarily an advantage, however, since mixtures that are easy to compact during

construction may continue to densify under traffic, ultimately leading to rutting due to low voids and plastic flow.

The surface texture, like particle shape, also influences the workability and strength of HMA. A rough, sandpaper–like surface texture, such as found on most crushed stones, tends to increase strength and requires additional asphalt cement to overcome the loss of workability, as compared to a smooth surface found in many river gravels and sands. Voids in a compacted mass of rough–textured aggregate are also usually higher, providing additional space for asphalt cement. Smooth–textured aggregates may be easier to coat with an asphalt film, but the asphalt cement usually forms stronger mechanical bonds with the rough–textured aggregates.

The combined effects of particle shape and surface texture of an aggregate can be determined using the particle index test of ASTM D3398. The equipment required for this test is simple, consisting basically of a cylindrical steel mold 6 inches (152mm) in diameter (diameter ranges from 2 inches or 51mm for particles passing No. 50 sieve to 8 inches or 203mm for particles larger than 1 inch or 25mm) by 7 inches (178mm) high, and a steel rod 5/8 inch (16mm) in diameter by 24 inches or 610mm long (the tamping end is rounded to a hemispherical tip). Tamping rods weigh between 34 g for the finest size fraction to 2,204 g for the largest size fraction. The procedure requires that a clean, washed, oven–dried, single–size aggregate fraction be used. The mold should be filled in three equal layers with each layer receiving ten tamps with the tamping rod. Each tamp consists of a drop with the tamping rod from 2 inches above the surface of the layer being compacted. The weight of the contents of the mold in each case is determined and the corresponding percentage of voids is calculated using the bulk specific gravity of each aggregate fraction. This procedure is repeated using the same material but applying 50 blows on each of the three layers. The particle index is then calculated using the following equation:

$$I_a = 1.25V_{10} - 0.25V_{50} - 32.0$$

where,

I_a = particle index value;

V_{10} = percent voids in the aggregate compacted with 10 blows per layer; and

V_{50} = percent voids in the aggregate compacted with 50 blows per layer.

The particle index of an aggregate containing several size fractions is weighted on the basis of the percentage of the fractions in the original grading of the aggregate. An aggregate composed of rounded particles with smooth surface textures may have a low particle index of 6 or 7, while aggregates consisting of highly angular crushed particles with rough surface textures can have particle indices of 15 to 20 or more. Figure 3–8 shows the effect of particle index on Marshall stability (*10*). This test is very time consuming and is therefore not widely used.

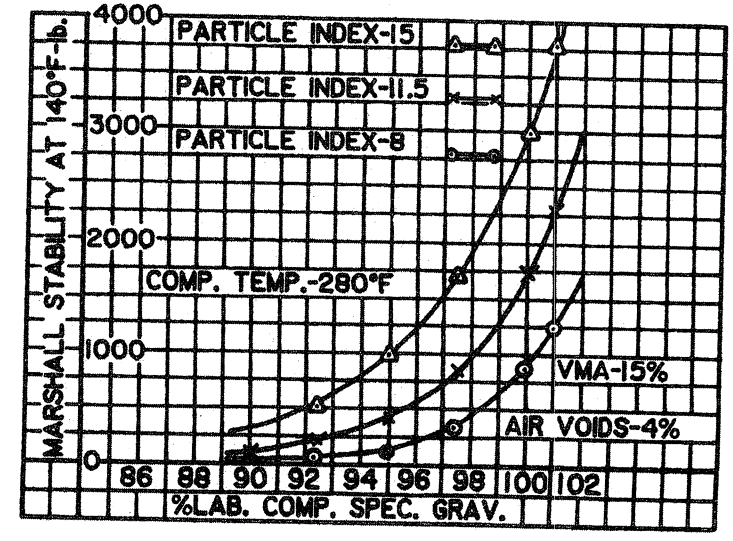


Figure 3–8. Effect of Particle Index on Marshall Stability (*10*)

An HMA can generally be made more stable and resistant to rutting by specifying that a significant portion (e.g., 90 percent) of the coarse aggregate have at least one fractured face. For high–type HMA, the

FHWA (*11*), in 1988, recommended that specifications require at least 60 percent of the plus No. 4 (4.75mm) material have at least two mechanically induced, fractured faces and that natural sand be limited to 15 to 25 percent. Natural sand is limited because it is generally rounded and hence is less desirable than crushed fine aggregates. Some agency specifications require that at least 75 percent or 85 percent of the coarse particles have two crushed faces and that at least 90 percent have one crushed face. Some agencies do not allow any natural sand in the mixes used for high volume roads such as interstate highways. The particle shape and texture of fine aggregates has more influence on the strength of fine graded HMA than that of coarse aggregate. A high percentage of fractured faces in the coarse aggregate is important for dense–graded mixtures to provide adequate stability but is even more important for open–graded friction courses. The standard test procedure for measuring the percentage of fractured faces for coarse aggregate is ASTM D5821.

The particle shape and surface texture of the fine aggregate can be quantified by the use of ASTM C1252 or AASHTO TP33 (Uncompacted Void Content of Fine aggregate, commonly called the fine aggregate angularity (FAA) test). This test has been shown to have potential for predicting the rutting potential of HMA based on the work done at the National Center for Asphalt Technology (*12*, *13*). This test has been included as a part of the Superpave mix design system. In this method, a 100–cm^3 cylinder is filled with fine aggregate of prescribed gradation by allowing the sample to flow through the orifice of a funnel into the calibrated cylinder. Excess material is struck off and the cylinder with aggregate is weighed. Uncompacted void content of the sample is then computed using this weight and the bulk dry specific gravity of the aggregate. Three variations of the method are discussed in the test method. Method A uses a graded sample of specified gradation. Method B uses three individual size fractions: Nos. 8 to 16 (2.36 to 1.18mm), Nos. 16 to 30 (1.18 to 0.6mm), and Nos. 30 to 50 (0.6 to 0.3mm); and the mean void content is determined. Method C uses the gradation as received. Method A is used in the Superpave mix design system.

Flat and elongated particles impede compaction and thus may prevent the development of satisfactory strength in HMA. These flat and elongated particles also break down during handling and mix production and can produce mixture volumetrics during plant production significantly different that that observed during mix design. Most state DOTs specify the flat and elongated count based on a 5:1 ratio or on a 3:1 ratio. The percentage of flat and elongated particles allowed varies between states. The flat and elongated particles in coarse aggregates can be determined following ASTM D4791, using a proportional caliper device.

Cleanliness and Deleterious Materials

Cleanliness refers to absence of certain foreign or deleterious materials that make aggregates undesirable for HMA. Washing the dirty aggregates can usually reduce the amount of undesirable foreign matter to an acceptable level. Typical objectionable materials include vegetation, shale, soft particles, clay lumps, clay coating on aggregate particles, and sometimes excess dust from the crushing operation. In addition to the petrographic examination described earlier, the following tests can be used to identify and measure the quantity of deleterious materials.

Sand Equivalent Test. The Sand Equivalent Test (ASTM D2419) is used to determine the relative proportions of plastic fines and mineral dust from fracture in fine aggregates. In this test, 85 ml of aggregate passing a No. 4 (4.76 mm) sieve is agitated in a water–filled transparent cylinder (1.25 inches or 32mm inside diameter, 17 inches or 432mm high, and graduated from the bottom up to 15 inches or 381mm by tenths) which is filled with a mixture of water and a flocculating agent (a combination of calcium chloride, glycerine and formaldehyde). After agitation and 20 minutes of settling, the sand separates from the flocculated clay, and the heights of clay and sand in the cylinder are measured. The sand equivalent is the ratio of the height of sand to the

total height of sand plus clay times 100. Cleaner aggregate will have a higher sand equivalent value. Specifications for aggregates in HMA often require that the results from the sand equivalent test be at least 25 to 35 or higher.

Clay Lumps and Friable Particles. Clay and friable particles (ASTM C142) may be present in the form of lumps. The harm caused by these lumps depends on their survival as lumps in HMA. These lumps in the finished HMA can break down from freezing and thawing or wetting and drying and cause stripping, raveling, or otherwise affect the durability of the HMA. Clay lumps and organics will often swell as they come in contact with moisture in the pavement resulting in popouts if located near the pavement surface. These particles can normally be removed during the aggregate crushing operation at the grizzley or in a washing operation. Some of the particles break down when processed through the HMA facility. Natural sands and gravels are the most likely source of these clay lumps and organics. If the lumps remain intact during placement of the HMA, surface pitting is likely to occur as the lumps on or near the surface deteriorate under rolling or traffic. If the total percent of clay lumps or friable particles in the HMA is very low (less than 1 percent), the surface problems will be minimal, and performance will be satisfactory. Some pitting and popouts may occur, but otherwise the performance will not be adversely affected.

In this test for clay lumps and friable particles, aggregates are first washed and dried to remove material passing the No. 200 (75micron) sieve and then separated into different sizes. Test samples are weighed and soaked in water for 24 hours. Any particles that can be broken with the fingers after soaking and removed by wet sieving are classified as clay lumps or friable particles, and the percent of this material is calculated by weight of the total test sample. Specifications normally limit the amount of clay lumps and friable particles to a maximum of 1 percent.

Plasticity Index. The Plasticity Index (PI) (ASTM D4318) is the difference between the liquid limit and the plastic limit of the material passing the No. 40 (0.42 mm) sieve. PI is a measure of the degree of plasticity of fines (material passing the No. 200 sieve) and can indirectly indicate the amount and type of plastic fines. ASTM D1073 and D242 limit the PI of the aggregate fraction passing the No. 40 sieve of both the fine aggregate and the mineral filler to four percent or less.

Specific Gravity

The specific gravity of an aggregate is useful in making weight–volume conversions and in calculating the void content in a compacted HMA. By definition, the specific gravity of an aggregate is the ratio of the weight of a unit volume of the material to the weight of an equal volume of water at approximately 23°C (73.4°F). One equation for specific gravity that is often used is:

$$Specific\ Gravity = \frac{weight}{volume(unit\ weight\ of\ water)}$$

When working in the metric system the unit weight of water is 1.0 gram per ml. Hence the equation for specific gravity becomes:

$$Specific\ Gravity = \frac{weight\ (grams)}{volume\ (ml)}$$

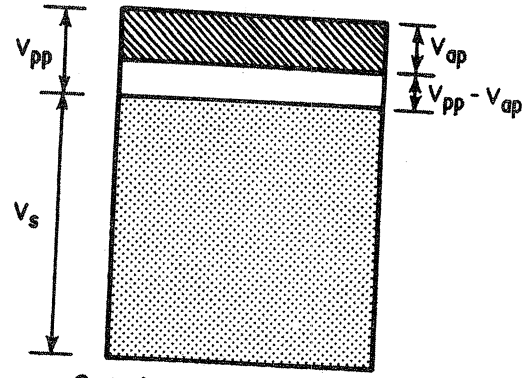
There are three different aggregate specific gravities used for HMA based on the method used to define the volume of the aggregate particles:

1. Apparent specific gravity;
2. Bulk specific gravity;
3. Effective specific gravity; and

Referring to Figure 3–9, the apparent specific gravity includes only the volume of the aggregate particle. It does not include the volume of any pores or capillaries that become filled with water after a 24–hour soaking. The bulk specific gravity includes the overall volume of the aggregate particle, as well as the volume of the pores that become filled with water after a 24–hour soaking. The effective specific gravity includes the overall volume of the aggregate, plus the pores that become filled with water after a 24–hour soaking, minus the volume of the larger pores that absorb asphalt. The volume of solids (V_s) contains internal pores that are impermeable to both water and asphalt. Since the volume of the impermeable pores cannot be accurately determined, for all practical purposes they are considered as part of the volume of the solids. Reducing the aggregate size by crushing exposes some of these internal voids and thus increases the apparent specific gravity of the aggregate.

Generally the effective specific gravity is equal to or higher than the bulk specific gravity and equal to or lower than the apparent specific gravity. When the aggregate absorption is zero, all three types of aggregate specific gravity are theoretically equal. However, due to normal testing variability the actual test results will typically be slightly different.

The bulk and apparent specific gravities of coarse and fine aggregates can be determined by ASTM C127 and C128, respectively. These tests are conducted on the aggregate particles. The effective specific gravity can be calculated from the theoretical maximum specific gravity (Rice specific gravity) using ASTM D2041. This test is conducted on loose asphalt mix so it is a very different approach than that used for the apparent specific gravity and the bulk specific gravity. The effective specific gravity will be discussed in more detail in Chapter 4.



Oven-dry weight of aggregate = W_s
Unit weight of water, γ_w = 1g/cm^3

(For simplicity, metric measurements are typically used in these calculations.)

V_s = Volume of solids
V_{pp} = Volume of water permeable pores
V_{ap} = Volume of pores absorbing asphalt
$V_{pp} - V_{ap}$ = Volume of water permeable pores not absorbing asphalt

$$\text{Apparent specific gravity} = G_{sa} = \frac{W_s}{V_s\ \gamma_w}$$

$$\text{Bulk specific gravity} = G_{sb} = \frac{W_s}{(V_s + V_{pp})\gamma_w}$$

$$\text{Effective specific gravity} = G_{se} = \frac{W_s}{(V_s + V_{pp} - V_{ap})\gamma_w}$$

Figure 3–9. Relationship between the Different Specific Gravities of an Aggregate Particle

Specific Gravity and Absorption of Coarse Aggregate. Equipment and procedures for determining the specific gravity and absorption of coarse aggregates are outlined in AASHTO T85 and ASTM C127. The method is briefly outlined here.

Approximately 5 kg of thoroughly washed aggregate retained on a No. 4 (4.75 mm) sieve is oven–dried to a constant weight. The dried sample is then immersed in water for 24 hours. The aggregate is removed

from the water, drained, and surface–dried with a cloth until all visible films of water are removed. The surfaces will still appear damp. The weight of the sample in the surface–dry condition is then obtained and recorded as B. The saturated surface–dry sample is placed in a wire basket, submerged in water, and the submerged weight determined and recorded as C. The sample is then removed from the water, drained and placed in an oven and dried to a constant weight. The oven–dried weight is recorded as A. The specific gravity and absorption are calculated as follows:

$$\text{Apparent specific gravity, } G_{sa} = \frac{A}{C} \quad (1)$$

$$\text{Bulk specific gravity } G_{sb} = \frac{A}{B-C} \quad (2)$$

$$\text{Absorption, \%} = \frac{(B-A)100}{A} \quad (3)$$

where,

A= oven–dried weight of aggregate, g;

B= saturated surface–dry (SSD) weight of aggregate, g; and

C= submerged weight of aggregate in water, g.

Specific Gravity and Absorption of Fine Aggregate. Equipment and procedures for determining the specific gravity and absorption of fine aggregates are described in AASHTO T84 and ASTM C128. The procedure is briefly outlined here. The flask (pycnometer) to be used to measure specific gravity is filled with water and the weight recorded as B. Approximately 1,000 g of fine aggregate is oven–dried to a constant weight. The dried sample is then immersed in water for 24 hours. The fine aggregate is spread on a clean flat surface and exposed to a gently moving current of warm air until a saturated surface dry condition is achieved. A saturated surface–dry (SSD) condition is reached at the moisture content at which the lightly compacted material in a cone first slumps when the cone is removed. The aggregate has some cohesion as

long as there is surface moisture but has no cohesion when the surface moisture evaporates; hence, the fine aggregate slumps when the cone is removed. The cone test must be repeated several times to ensure that the highest moisture content (saturation) at which the aggregate will slump is determined. The SSD condition is sometimes difficult to accurately measure. Some state DOTs will wash out the dust (material passing the No. 200 sieve) before conducting the fine aggregate specific gravity test. This has been found to reduce the variability and give a more accurate test result.

Approximately a 500 g sample of the saturated surface dry material is placed in the flask and the actual weight of the sample recorded as D. The flask is then filled with water, using a specified procedure, and the weight recorded as C. The fine aggregate is removed from the flask, oven–dried to a constant weight, and the weight recorded as A. Specific gravity and absorption are calculated as follows:

$$\text{Apparent specific gravity, } G_{sa} = \frac{A}{B+A-C} \quad (4)$$

$$\text{Bulk specific gravity, } G_{sb} = \frac{A}{B+D-C} \quad (5)$$

$$\text{Absorption, \%} = \frac{D-A}{A} \times 100 \quad (6)$$

where,

A = weight of oven–dry sample, g;

B = weight of flask (pycnometer) filled with water, g;

C = weight of flask (pycnometer) with specimen and water to calibration mark, g; and

D = saturated surface dry weight (approximately 500 grams).

When the sample is tested in separate size fractions (for instance, coarse and fine), the average specific gravity value can be computed as

the weighted average value of the various fractions using the following equation:

$$G = \frac{P_1 + P_2 + \ldots + P_n}{\frac{P_1}{G_1} + \frac{P_2}{G_2} + \ldots + \frac{P_n}{G_n}} \qquad (7)$$

where,

G = average specific gravity;

$G_1, G_2, \ldots G_n$ = specific gravity values for fraction 1, 2,...n; and

$P_1, P_2, \ldots P_n$ = weight percentages of fraction 1, 2,...n.

This same equation can be used to calculate the average specific gravity of an aggregate blend in which case $G_1, G_2, \ldots G_n$ are the specific gravities of each of the aggregates 1, 2,...n and $P_1, P_2, \ldots P_n$ are the weight percentages of each aggregate 1, 2,...n in the blend. This equation can be used to determine average bulk specific gravity or average apparent specific gravity, but the specific gravity type must be the same for all aggregates being blended.

Example 3–1. The apparent specific gravity of an aggregate is 2.750 and the water absorption is 2.0 percent. Determine the bulk specific gravity.

Recall that

$$Gsa = \frac{dry\ weight\ of\ aggregate}{volume\ of\ aggregate}$$

when the metric system is used and

$$Gsb = \frac{dry\ weight\ of\ aggregate}{volume\ of\ aggregate\ \&\ external\ pores}$$

where,

G_{sa} = apparent specific gravity; and

G_{sb} = bulk specific gravity.

The volume of the external pores is equal to the volume of absorbed water when the aggregate is in SSD condition. Assume for this example that the weight of dry aggregate is 100 grams. Then

volume of water (ml) = weight of water (grams)

$$Water\ content = \frac{weight\ of\ water}{dry\ weight\ of\ solids}$$

The water content in the saturated surface dry condition is 2.0 percent as given in the problem. The aggregate must be in the SSD condition to measure all of the external pores.

weight of water in SSD condition = 0.02 (100) = 2 grams

volume of water in SSD condition = 2 ml, since 1 gram of water = 1 ml volume

Hence, the volume of external voids is 2 ml. Therefore

$$Volume\ of\ aggregate = \frac{dry\ weight\ of\ aggregate}{Gsa} = \frac{100}{2.750} = 36.364\ ml$$

it then follows that

$$G_{sb} = 100/(36.364 + 2.000) = 100/38.364 = 2.607$$

Example 3–2. Two aggregates will be combined in equal proportions by weight to meet a HMA specification. The bulk specific gravity of aggregate #1 is 2.500, and aggregate #2 is 3.000. Determine the bulk specific gravity of the blend.

This problem is worked two ways. The equation for determining the average specific gravity of a blend of aggregate is:

$$G = \frac{P1 + P2}{\frac{P1}{G1} + \frac{P2}{G2}}$$

Substituting into the equation above

$$G = \frac{0.500 + 0.500}{\frac{0.500}{2.500} + \frac{0.500}{3.000}} = \frac{1}{0.200 + 0.167}$$

$$= \frac{1}{0.367}$$

Hence,

$$G = 2.725$$

The problem will now be worked by simply dealing with weights and volumes. The two aggregates are combined by equal proportions by weight. Assume the weight of aggregate #1 is 100 grams. Hence, the weight of aggregate #2 is also 100 grams.
The volume of aggregate #l is determined as shown below:

Volume of aggregate 1 = (weight of aggregate 1)/ (specific gravity of aggregate 1)

$$= \frac{100}{2.500}$$

$$= 40.000$$

The volume of aggregate #2 is determined in a similar manner:

Volume of aggregate 2 = (weight of aggregate 2)/(specific gravity of aggregate 2)

$$= \frac{100}{3.000}$$

$$= 33.333$$

Hence, the total weight of the blended aggregate is 200 grams and the total volume of the blended aggregate is 73.333 ml. It follows that

$$G_{sb} = \frac{weight}{volume}$$

$$= \frac{200}{73.333}$$

$$= 2.727$$

Simply averaging the two specific gravities does not provide the correct combined specific gravity but in this case it is close enough to be within the rounding error.

Size and Gradation

Desired Gradation Curves. Aggregate gradation is the distribution of particle sizes expressed as a percent of the total weight. The gradation as a percent of the total volume is of most importance, but expressing gradation as a percent by weight is much easier to do and is standard practice in the industry. The gradation by volume and by weight is approximately the same as long as the specific gravities of the various aggregates being used are approximately equal. If there are large differences in the specific gravity of aggregates being used for a particular mix, then the gradation should be determined as a percent of total volume. Gradation is determined by sieve analysis, that is, by passing the material through a series of sieves stacked with progressively smaller openings from top to bottom, and weighing the material retained on each sieve. The gradation of an aggregate is normally expressed as total percent passing various sieve sizes.

Gradation of an aggregate can be graphically represented by a gradation curve for which the ordinate is the total percent by weight passing a given size on an arithmetic scale, while the abscissa is the particle size plotted to a logarithmic scale. Sieves typically used for sieve analysis and gradation specifications for HMA are: 2 inches, 1 1/2

inches, 1 inch, 3/4 inch, 1/2 inch, 3/8 inch, No. 4, No. 8, No. 16, No. 30, No. 50, No. 100, and No. 200 (50.8mm, 38mm, 25.4mm, 19mm, 12.5mm, 9.5mm, 4.75mm, 2.36mm, 1.18mm, 0.6mm, 0.3mm, 0.15mm, and 0.075mm, respectively). A 3/8–inch sieve has openings equal to 3/8 inch. On the other hand a No. 8 sieve has 8 openings per inch. The openings in a No. 8 sieve are smaller than 1/8 inch since the diameter of the wire must be considered in determining the opening size. The sizes of successive sieves usually differ by a factor of approximately 2; therefore, when plotted on a logarithmic scale, the distances between adjacent sieve sizes are usually about equal. Aggregate gradations are sometimes described as dense or well–graded, uniformly–graded (open), and gap–graded as shown in Figure 3–10. For dense graded mixtures the terms coarse-graded and fine-graded are often used to distinguish whether the combined gradation is on the coarse side or the fine side of the maximum density line. This will be discussed in more detail in Chapter 4.

Gradation is certainly one of the most important properties of an aggregate. It affects almost all the important properties of an HMA, including stiffness, stability, durability, permeability, workability, fatigue resistance, friction, and resistance to moisture damage. Therefore, gradation is a primary consideration in asphalt mix design, and state specifications place limits on the percentage of various size fractions that can be used in HMA. States generally use a range of mixes with varying aggregate gradation requirements depending on traffic, location in pavement structure, etc.

Theoretically, it would seem reasonable that the best gradation for HMA is one that gives the densest particle packing. The gradation having maximum density provides increased stability through increased interparticle contacts and reduced voids in the mineral aggregate. However, there must be sufficient air void space to permit enough asphalt cement to be incorporated to ensure durability, while still leaving some air space in the mixture to avoid bleeding and/or rutting. A tightly packed aggregate (low voids in mineral aggregate) also results in a mixture that is more sensitive to slight changes in asphalt content.

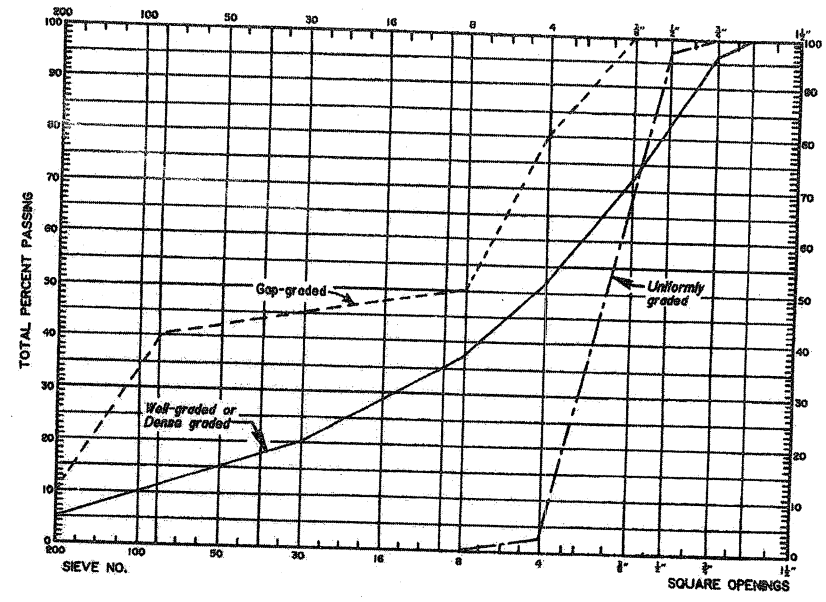


Figure 3–10. Typical Terms Used to Identify Aggregate Gradations

Numerous investigators have proposed ideal gradations for maximum density. One of the best known of these gradations is Fuller's curve proposed by Fuller and Thompson (*14*). The equation for Fuller's maximum density curve is:

$$P = 100(d/D)^n \qquad (8)$$

where d is the diameter of the sieve size in question, P is the total percent passing or finer than the sieve, and D is the maximum size of the aggregate. Studies by Fuller and Thompson showed that a maximum density can be obtained for an aggregate when n=0.5.

In the early 1960s, the Federal Highway Administration introduced an aggregate grading chart which is based on the Fuller gradation but uses a 0.45 exponent in the equation. This chart is very convenient for determining the maximum density line and for adjusting aggregate gradings. Using this chart, the maximum density lines can be easily

obtained by drawing a straight line from the origin at the lower left of the chart to the actual percentage point of the nominal maximum size. The nominal maximum size is defined as the largest sieve size in the specification upon which any material is retained. Figure 3–11 shows a family of maximum density lines plotted on this chart. FHWA recommends this chart be used as part of the HMA design process.

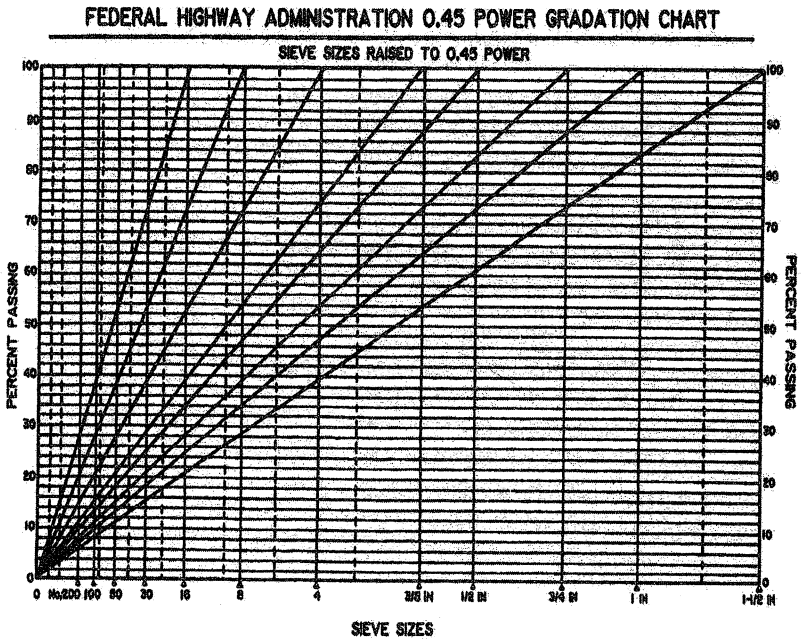

FEDERAL HIGHWAY ADMINISTRATION 0.45 POWER GRADATION CHART
SIEVE SIZES RAISED TO 0.45 POWER
PERCENT PASSING
PERCENT PASSING
0 No.200 100 50 30 16 8 4 3/8 IN 1/2 IN 3/4 IN 1 IN 1-1/2 IN
SIEVE SIZES

Figure 3–11. Maximum Density Lines for FHWA 0.45 Power Gradation Chart

Gradations of maximum density may not provide sufficient voids in the aggregate for enough asphalt cement to provide adequate film thickness for maximum durability without bleeding. In such cases deviations from the maximum density curves are necessary in order to increase the total voids in the mineral aggregate (VMA). Minimum VMA requirements have been suggested and vary with the maximum nominal aggregate size (see Chapter 4). Most asphalt paving agencies

prefer that the gradation be approximately parallel to the maximum density grading, but that it be offset a few percentage points either above or below the line. The actual gradation and the maximum density line will intersect at the nominal maximum size and be a few percentage points different on the middle sieves. The requirement for minimum VMA is necessary to ensure that there are sufficient voids in the aggregate to allow enough asphalt cement to be added to provide a durable mix and sufficient air voids to maintain stability. The Superpave mix design developed by the Strategic Highway Research Program (SHRP) requires a selected number of control points on the gradation chart (see Chapter 4).

Maximum Aggregate Size. The maximum particle size used in a mixture is important to ensure good performance. If the maximum particle size is too small, the mix may be unstable; if it is too large, workability and segregation may be problems. There are two designations for maximum particle size (ASTM C125):

1. Maximum size, designated as the smallest sieve through which 100 percent of the aggregate sample particles pass; and
2. Nominal maximum size, designated as the largest sieve that retains some aggregate particles, but not more than 10 percent of the aggregate.

Mix designations used in specifications typically use the nominal maximum size of aggregate. For example, in ASTM specification D3515, a blend having nominal maximum aggregate size of 1 inch (25.4mm) is required to have 100 percent of the material passing the 1 1/2–inch (38mm) sieve and 90–100 percent passing the 1–inch sieve (25.4mm).

Superpave mix design system uses the following aggregate size definitions:

Maximum Size: One sieve size larger than the nominal maximum size.

Nominal Maximum Size: One sieve size larger than the first sieve to retain more than 10 percent.

The maximum density line is obtained in Superpave by connecting the origin at the lower left of the 0.45 power gradation chart, as shown in Figure 3-12, to the maximum aggregate size at the upper right of the chart (*15*).

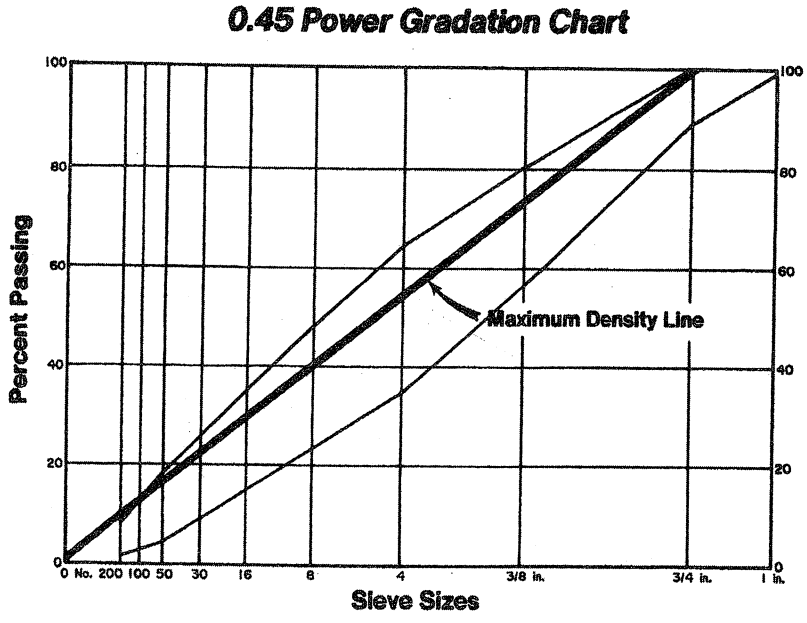


Figure 3–12. ASTM D 3515 Specification for 3/4–inch Nominal Maximum Aggregate Size Plotted on the 0.45 Power Chart

From a construction standpoint, the maximum aggregate size is normally limited to about one–half of the lift thickness. In recent years, there has been an increase in the use of large stone mixes to minimize rutting potential of HMA. The introduction of larger stone sizes increases the volume concentration of the aggregate. This factor contributes to a reduction of both design asphalt content and cost of the mix. These large stone mixtures are more resistant to rutting than the smaller aggregate

size mixtures. However, the use of a maximum aggregate size greater than 1 inch (25.4mm) often results in harsh mixes that tend to segregate during placement. Therefore, special attention must be given to mix design, mat thickness, material handling, mixing, and paving procedures when these larger maximum aggregate size mixtures are used.

Typical Specification Bands and Tolerances. Aggregate gradation specifications for HMA have been developed through accumulated field experiences. In many cases they are established by trial and error to reflect local conditions. Most specifications for HMA require well or dense–graded aggregate gradations with the middle portion of the curves approximately parallel to the maximum density curves. ASTM D 3515 recommend gradation limits and tolerances for a number of nominal maximum aggregate sizes. In practice, most state specifications do not allow as much variation in gradation as that provided in ASTM D 3515. The gradation band from ASTM D3515 for 3/4–inch (19mm) nominal maximum size mixture is plotted on a FHWA gradation chart in Figure 3–12. For the most part, the maximum density curve for a 3/4–inch (19mm) maximum size aggregate fits inside these limits. Some guidance for developing a gradation band and potential problem areas are shown in Figure 3–13 (*16*).

Another problem often caused by poor aggregate gradation is tender mixes. These mixtures cannot be compacted in the normal manner because they are slow in developing sufficient stability to withstand the weight of the compaction equipment and they tend to move underneath the roller during compaction. Figure 3–14 illustrates a typical gradation pattern for these mixes. Most notable is the hump in the curve near the No. 40 sieve and the flat slope between the No. 40 and the No. 8 sieves. The most likely cause of an aggregate blend having this shape curve is excessive use of poorly–graded natural sands. So it is not clear if it is the humped gradation that causes the tender mix problem or if it is the natural sand that typically causes the humped gradation that caused the tender mix problem. There are other causes of tender mixtures which are discussed in Chapter 6.

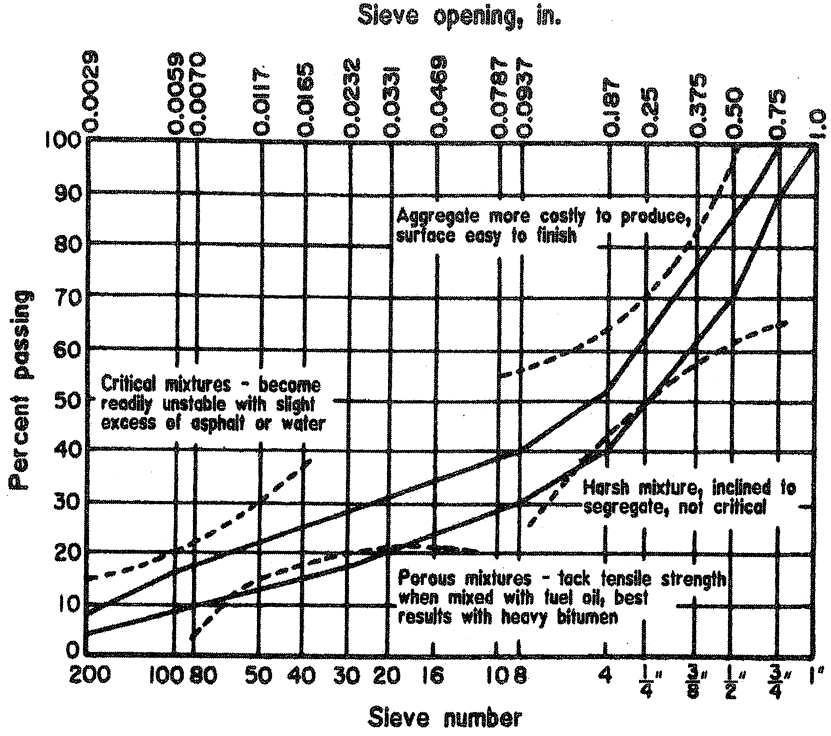


Figure 3–13. Gradation Limits for HMA and Expected Problems when Outside the Limits (*16*)

Sieve Analysis (ASTM Cl36 and C117). The gradation of an aggregate is determined by a sieve analysis. Standard procedures for a dry sieve analysis are given in ASTM C136, and for a washed–sieve analysis for determining the amount of material passing the No. 200 (75 microns) sieve, the procedures are in ASTM C117. The washed sieve analysis is a more accurate measure of the true gradation, but the dry method is faster and is often used to estimate the actual gradation. When the dry method is used, the measured amount of material passing the No. 200 (75 microns) sieve is most likely significantly lower than the actual amount in the aggregate mixture. For clean aggregate, a dry sieve analysis may be accurate enough since the amount of material passing the No. 200 (75 microns) sieve is low.

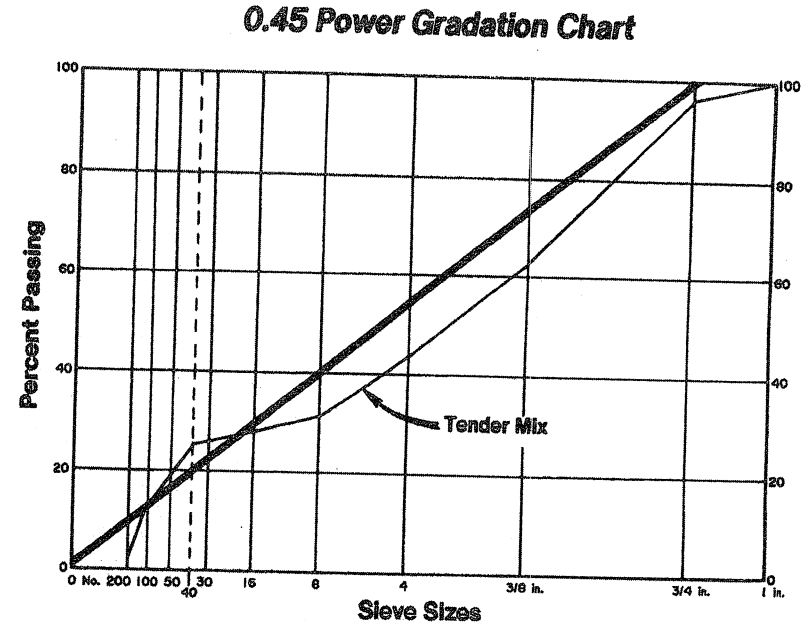


Figure 3–14. Characteristics of a Tender Mix on the 0.45 Power Gradation Chart

Example 3–3. Assume that an aggregate blend was split into two samples. Determine the percent passing each sieve size using the dry and washed gradation methods. A dry gradation was conducted on the first sample which weighed 2012 grams (Table 3–7). A washed gradation was performed on the second sample which weighed 2074 grams. The weights and calculations for the washed gradation are shown in Table 3–8.

The steps in a dry sieve analysis include splitting the sample to the correct size, determining the total dry weight, pouring it over a set of sieves and shaking, weighing the material collected on each sieve size, determining the percent retained on each sieve, and determining the percent passing each sieve. When a washed sieve analysis is conducted to obtain the washed gradation, the material is washed over a No. 200 (75 micron) sieve to remove the dust. The material is then dried,

weighed, poured over a nest of sieves, vibrated, and the amount retained on each sieve determined.

Table 3–7. Computation of Dry Aggregate Gradation

Sieve Size	Weight Retained	Percent Retained	Percent Passing
3/4 inch	0.0	0.0	100.0
1/2 inch	86.5	4.3	95.7
3/8 inch	154.9	7.7	88.0
No. 4	253.5	12.6	75.4
No. 8	269.6	13.4	62.0
No. 16	344.1	17.1	44.9
No. 30	380.3	18.9	26.0
No. 50	323.9	16.1	9.9
No. 100	158.9	7.9	2.0
No. 200	22.1	1.1	0.9
Pan	18.2	0.9	
Total	2012.0	100.0	

Aggregate Blending to Meet Specifications. For various reasons, mostly associated with achieving maximum density and desired void properties, certain desirable gradation limits are usually required of aggregates for HMA. Because it is unlikely that a single natural or quarried material will meet these specifications, two or more aggregates of different gradations are typically blended to meet specifications limits. Even if one aggregate source could be used to meet the specifications this would not be an acceptable method since segregation would be a major problem if one tried to stockpile all aggregate sizes in one stockpile. Mixing of coarse and fine aggregate in one stockpile results in segregation; hence, aggregates should be separated into sizes, for example, 3/4 inch–3/8 inch (19mm–9.5mm), 3/8 inch–No. 4 (9.5mm–4.75mm), and minus 4 (4.75mm) prior to hauling and stockpiling.

Another reason for blending aggregates is that it is often more economical to combine naturally occurring and processed materials to meet specifications than to use all processed materials.

Table 3–8. Computation of Washed Aggregate Gradation

Sieve Size	Weight Retained	Percent Retained	Percent Passing
3/4 inch	0	0	100
1/2 inch	89.2	4.3	95.7
3/8 inch	159.7	7.7	88.0
No. 4	238.5	11.5	76.5
No. 8	311.1	15.0	61.5
No. 16	300.7	14.5	47.0
No. 30	398.2	19.2	27.8
No. 50	327.7	15.8	12.0
No. 100	149.3	7.2	4.8
No. 200	16.6	0.8	4.0
Pan	24.9	1.2	
Washed Out	58.1	2.8	
Total	2074.0	100.0	

The nature of particle size distribution can be examined by graphically representing the gradation by (a) a cumulative percent passing on a semi-log scale, or (b) the cumulative percent passing versus the sieve sizes raised to the 0.45 power. Both methods are commonly used by engineers to help select the best aggregate blend.

If the specifications are expressed in terms of the total percent passing each sieve, they can be plotted as bands or envelopes (Figure 3–15). A method of transforming a passing–retained specification to an approximate equivalent total percent passing specification was described by Dalhouse (*17*). The transformation enables the passing–retained

specification to be plotted on the total percent passing chart, making visual examinations and comparisons of different specifications possible. However, in this chapter all gradations and specifications are expressed on a total percent passing basis, unless otherwise stated.

A large number of blending methods (techniques of determining relative proportions of various aggregates to obtain a desired gradation) have been developed since the suggestion of the maximum density curves by Fuller and Thompson (*14*). The suitability of these methods depends on the types of specifications and number of aggregates involved, the experience of the individual, and the major emphasis of the blending (closeness to the desired gradation or economics).

Regardless of the number of aggregates or blending techniques selected, the basic formula expressing the combination is

$$p = Aa + Bb + Cc + \ldots \quad (9)$$

where,

p = the percent of material passing a given sieve for the combined aggregates A, B, C,.....

A, B, C,... = the percent of material passing a given sieve for each aggregate A, B, C,...

a, b, c,... = proportions (decimal fractions) of aggregates A, B, C,... to be used in the blend, a + b + c... = 1.00

It is desirable, no matter which method is used, to first plot the gradations of the aggregates to be blended and the specification limits on a gradation chart before actual blending is attempted. From these plots, decisions can be made prior to any calculation on (a) whether a blend can be found using the available aggregates to meet the specification limits, (b) where the critical sieve sizes are, and (c) the approximate trial proportions to be selected. These decisions can be made based on the following simple facts and the gradations shown in Figure 3–15:

1. The gradation curves for all possible combinations of aggregates A and B fall between curves A and B. It is impossible to blend aggregates C and B to meet the specification regardless of the method used.
2. If two curves cross at any point (B and D), the grading curves for all possible combinations pass through that point.
3. The curve for a blend containing more of aggregate A than B is closer to curve A than B and vice versa.

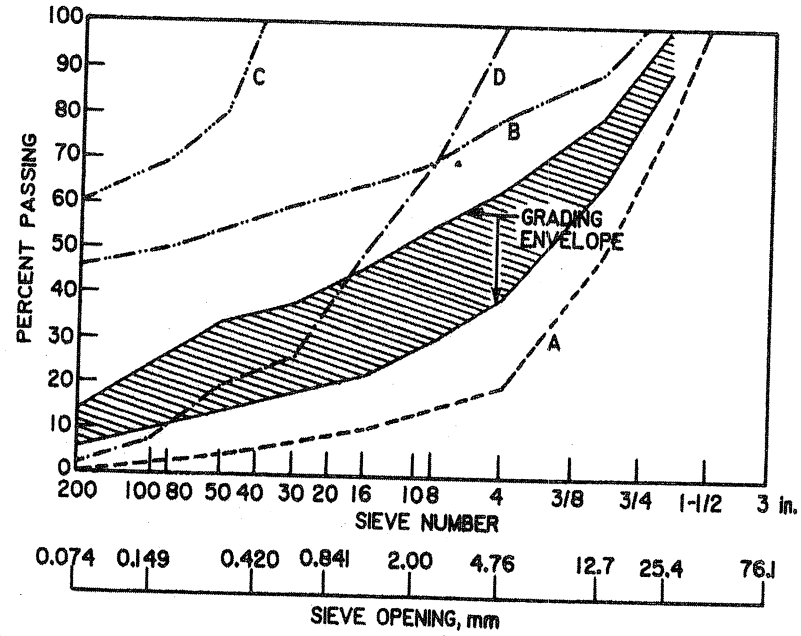


Figure 3–15. Examination of Aggregate Gradings with Specification Band

The most common method of determining the proportions of aggregate to use to meet specification requirements is the trial–and–error method. As the name implies, a trial blend is selected (aided by experience and plots of individual gradation curves and specification limits) and calculations made using Equation 9 to determine the percent passing each sieve size for the blend. The grading that is calculated from this trial is compared with the specification requirements. Adjustments are made for the second trial blend and the calculations repeated for the

critical sieves until a satisfactory or optimum blend is obtained. This method, guided by a certain amount of reasoning, mathematics, and experience, is the most widely used method and is the easiest procedure to determine a satisfactory blend.

The trial–and–error method involves the following steps:

1. Selecting critical sieves for the aggregates in the blend;
2. Determining an initial set of proportions a, b, c, etc., which will meet the specification requirements for the critical sieves;
3. Checking the calculated blend using the proportions determined for all sieves in the specification requirements; and
4. Adjusting the proportions, as necessary, to ensure that the percentages for all sieves are within specification limits.

Example 3–4. Determine a blend of the two aggregates shown in Table 3–9 which will meet specification requirements using the trial–and–error method.

Examination of the grading indicates that it is possible to find a blend that falls within the specification limits, possibly a 50–50 blend because of the relative distance of the curves to the center of the band (desired gradation). The first trial blend can be determined more intelligently if certain critical sieve sizes are selected. By inspecting the gradations, it is seen that all material retained on the 3/8–inch or 9.5mm sieve (100–80 = 20 percent desired) has to come from aggregate A, and all material finer than the No. 30 (0.6mm) sieve must be furnished by aggregate B. With regard to aggregate A (since 100–59 = 41 percent is retained on the 3/8–inch (9.5mm) sieve from A), the percentage needed from A to retain 20 percent on this sieve (specification median) is A = 20/41 = 0.49. The percentage of aggregate B will then be 1–0.49 = 0.51. With regard to aggregate B (since there is 51 percent passing the No. 30 (0.6mm)), the percentage of aggregate B required to arrive at the desired 24 percent passing for this sieve is B = 24/51 = 0.47. The percent of

aggregate A for this blend will be 1–0.47 = 0.53. Hence, analysis of the two critical sieves produces very similar results.

Evaluation of one of the critical sieves shows that the percent of aggregate A should be 49 percent and the other critical sieve shows that the percent of aggregate A should be 53 percent. For the first trial, try 50 percent of aggregate A and 50 percent of aggregate B.

A form that is easy to use is shown in Table 3–9. Multiply 0.50 times the percent passing each sieve size for aggregate A and aggregate B. Add the two results to get the gradation of the trial blend. The resulting gradation is compared to the desired one and it appears that the use of 50 percent A and 50 percent B results in a gradation very close to the desired gradation. Hence, for this example, the proportions should be 50 percent A and 50 percent B. The percentage of these two aggregates could be changed slightly, if desired, without having an adverse effect on the blend. For instance, if aggregate A costs more than aggregate B, the percentages could be changed to slightly increase the amount of aggregate B and decrease the amount of aggregate A, thus reducing the overall cost of the mix. The project specification will still be met but with a less expensive mix.

Table 3–9. Blending of Two Aggregates

Sieve Size										
Aggregate	**Percent Used**	**3/4 in**	**1/2 in**	**3/8 in**	**No. 4**	**No. 8**	**No. 30**	**No. 50**	**No. 100**	**No. 200**
Aggregate A	100	100	90	59	16	3	0	0	0	0
Aggregate B	100	100	100	100	96	82	51	36	21	9
50 percent A	50	50	45	29.5	8	1.5	0	0	0	0
50 percent B	50	50	50	50	48	41	25.5	18	10.5	4.5
Blend	100	100	95	79.5	56	42.5	25.5	18	10.5	4.5
Desired	100	100	90	80	60	43	24	18	12	7
Specification		100	80-100	70-90	50-70	35-50	18-29	13-23	8-16	4-10

Example 3–5. Given the gradation of aggregates A, B, and C, determine the required percent of each to result in a blend meeting the required specification requirements (see Table 3–10).

The critical sieves in this case would be the No. 4 (4.75mm) and No. 30 (0.6mm). The No. 200 (75 microns) sieve is also critical since aggregate C has 88 percent passing the No. 200 (75 microns) sieve and the specifications only allow up to 11 percent. The desired 52 percent of material larger than the No. 4 (4.75mm) sieve must come from aggregate A. Hence, the percent of aggregate A to use for the first trial should be:

$$percent\ of\ A = 52/81 = 64\%$$

Now looking at the No. 30 (0.6mm) sieve it is apparent that 75 percent of the material larger than a No. 30 (0.6 mm) sieve must come from materials A and B. Since 75 percent is the total desired to be retained on the No. 30 (0.6mm) and 0.64(95) is that portion provided by aggregate A.

$$\begin{aligned} percent\ of\ B &= 75 - 64\ (95) \\ &= 75 - 61 \\ &= 14\% \end{aligned}$$

Based on these calculations, it appears that the best first estimate would be 64 percent of aggregate A, 14 percent of aggregate B, and 22 percent of aggregate C. It is obvious, however, that using 22 percent of aggregate C would produce too much material passing the No. 200 (75 microns) sieve. The percent of aggregate C should probably be no more than 8 percent, since that aggregate alone will produce approximately 7 percent passing the No. 200 (75 microns) sieve. Based on this analysis, the first estimate should be 71 percent of aggregate A, 21 percent of aggregate B, and 8 percent of aggregate C. The calculations are shown in Table 3–11.

Table 3–10. Blending of Three Aggregates

Sieve Size	Aggregate A	Aggregate B	Aggregate C	Specification	Median of Specifications
1 inch	100	100	100	94 to 100	97
½ inch	63	100	100	70 to 85	78
No. 4	19	100	100	40 to 55	48
No. 8	8	93	100	30 to 42	36
No. 30	5	55	100	20 to 30	25
No. 100	3	36	97	12 to 22	17
No. 200	2	3	88	5 to 11	8

The estimated percentages of 71 percent A, 21 percent B, and 8 percent C produce a blend that meets the specification requirements. However, the blend can be improved slightly to get it closer to the desired gradation. To improve the blend, use 2 percent less of aggregate C and 5 percent less of aggregate A and, hence, 7 percent more of aggregate B. For each 1 percent that two aggregates are changed the resulting blend will change by the difference between the percent passing for any given sieve size for the two aggregates times 1 percent. For instance, increasing aggregate B by 5 percent and decreasing aggregate A by 5 percent will result in a change of 0.81 x 5 or 4.0 percent on the No. 4 sieve.

After making the calculations (Table 3–11) it can be seen that the amount of each aggregate to use is 66 percent A, 28 percent B, and 6 percent C.

The computations required by the trial–and–error method shown in the example were carried out manually to show the steps. However, successive trials can be done easily and quickly by using personal computers and electronic spreadsheet software. If desired, the gradations

of the blends as well as the specification band can be plotted and examined using the graphic capabilities of a typical personal computer.

Table 3–11. Calculations for Blending Three Aggregates

Aggregate	Percent Used	Sieve Size						
		1"	1/2"	No. 4	No. 8	No. 30	No. 100	No. 200
A	71	71	44.7	13.5	5.7	3.6	2.1	1.4
B	21	21	21	21	19.5	11.6	7.6	0.6
C	8	8	8	8	8	8	7.8	7.0
Blend		100	73.7	42.5	33.2	23.2	17.5	9.0
Desired		97	78	48	36	25	17	8.0
Specification		94-100	70-85	40-55	30-42	20-30	12-22	5-11
A	66	66	41.6	12.5	5.3	3.2	2.0	1.3
B	28	28	28	28	26.0	15.4	10.1	.08
C	6	6	6	6	6	6	5.8	5.3
Blend	100	100	75.6	46.5	37.3	24.6	17.9	7.4

There are several graphical methods that have been used to determine the percent of each aggregate to use in a blend. These methods are generally not used since the trial–and–error method is quicker and easier in most cases. These methods are especially difficult to use when more than two aggregates are being blended, which is normally the case for HMA. For more information on graphical methods, additional references should be reviewed (*6*, *18*, *20*, *21*).

Computer programs have been developed to determine the optimum blend for a specified number of aggregates. None of these programs have been very successful since an experienced engineer can determine the desired proportion of up to four aggregates by the trial–and–error method by the time he could input the data into the computer. Another problem with a computer method is that it is difficult to quantitatively describe the optimum blend. For example, the significance of a 2 percent deviation

from the desired gradation is not as critical for a No. 4 (4.75mm) sieve as for a number 200 (75 microns) sieve. Many factors must be considered when evaluating the importance of deviations from the desired blend. Some engineering judgment must be used, and one program cannot be developed that includes all the factors that must be considered when blending aggregates. For more information on computer methods, additional references should be reviewed (*19*, *21*, *22*). The most common use of computers in aggregate blending is in the trial–and–error method (using the engineer's judgment for the best blend) and in plotting the specifications and trial blends. Computer programs can help to factor in cost considerations (*23*, *24*, *25*).

In practice there is a trend to use a larger number of aggregate stockpiles to produce the desired gradation. Using more stockpiles will typically make it easier to meet any desired gradation. During production using more stockpiles will reduce variability in the final blend that occurs when the gradation in one of the stockpiles changes. The net result is less variability in the final blend.

Surface Area

One important property that can be computed from the aggregate gradation is the surface area. The aggregate surface area is important since it affects the amount of asphalt needed to coat the aggregate. Dense–graded asphalt mixtures are usually designed to contain a desired amount of air voids; hence, the aggregate surface area is not a factor in the design of the mixture. It is possible to increase the surface area of an aggregate and at the same time reduce the optimum asphalt content. One way to do this is by increasing the dust content of a mixture. The dust increases the surface area but at the same time results in a reduction in voids, so the increased dust will actually result in a decrease in optimum asphalt content even though it also results in an increase in surface area. Asphalt mixes that have high surface area and low optimum asphalt content are undesirable because these mixes will have a thin asphalt film

on the aggregate and will probably not have adequate durability. A further discussion of asphalt film thickness is provided in Chapter 4.

The most common method used for estimating the aggregate surface area involves calculations made from the aggregate gradation. The surface area is determined by multiplying surface area factors by the percentage passing the various sieve sizes. These factors can be calculated by assuming an aggregate specific gravity and assuming all particles are rounded or cube shaped. The surface area per pound of aggregate can then be determined for each aggregate size.

The surface area factors for various sieve sizes are shown in Table 3–12. The surface area is determined by multiplying the factor by the percentage passing for each sieve size and totaling for all sieve sizes. The units for these calculations are square feet per pound. All surface area factors for all sieve sizes shown in Table 3–12 must be used. The surface area is only slightly affected by the percent passing the larger sieve sizes but is significantly affected by the percent passing the smaller sieve sizes.

One of the primary reasons for estimating the surface area is to determine the asphalt film thickness. This is an estimated value, but it does allow comparisons to be made for various mixtures.

Table 3–12. Surface Area Factors

Sieve Size	Surface Area Factors
Percent Passing Maximum Sieve Size	2
Percent Passing No. 4	2
Percent Passing No. 8	4
Percent Passing No. 16	8
Percent Passing No. 30	14
Percent Passing No. 50	30
Percent Passing No. l00	60
Percent Passing No. 200	160

Example 3–6. An asphalt mixture has an aggregate gradation as shown below. Determine the surface area per pound of aggregate.

Sieve Size	Percent Passing
3/4 inch	100
1/2 inch	88
3/8 inch	72
No. 4	60
No.8	47
No. 16	35
No. 30	25
No. 50	20
No. 100	10
No. 200	5

Using the surface area factors and the percent passing each sieve shown above, the surface area (SA) is calculated below:

$$SA = 2(100) + 2(.60) + 4(.47) + 8(\sim 35) + 14(.25) + 30(.20) + 60(.10) + 160(.05)$$

Simplifying the results,

SA = 31.38 square feet of surface area per pound of aggregate.

An additional discussion of aggregate issues is provided in the National Stone Association Aggregate Handbook (*26*).

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CHAPTER 4. HOT MIX ASPHALT MIX DESIGN METHODOLOGY

INTRODUCTION

In 1995, a few states began to use the Superpave design procedures for a small number of projects. By 2008 most states had fully adopted Superpave as their mix design procedure. Superpave has become the mix design of choice for highway pavements in the US and by 2008 it was beginning to be adopted by airfields.

Most HMA produced during the 50 years between the 1940s and mid 1990s were designed using the Marshall or Hveem methods. According to a survey in 1984, approximately 75 percent of the state highway departments used some variation of the Marshall method while the remaining 25 percent used some variation of the Hveem method (*1*). Some states used both methods on the same materials to get a better understanding of the characteristics of the mixtures. It is important to recognize that all three methods (Hveem, Marshall, and Superpave) are designed to help the asphalt technologist choose an appropriate asphalt content with which to begin field construction. If the service conditions for which a mixture is being designed are different from those assumed during the mix design, then the design may not be adequate for good performance under the existing conditions. It is incumbent upon the user of each of these methods to understand the limitations of that design method to ensure that the method is used appropriately. To enhance that understanding, these three mixture design methods are discussed in this chapter. The focus of this chapter is on dense graded HMA mixtures.

HISTORY OF MIX DESIGN

Crawford has traced the history of asphalt mix design dating back to the 1860s and has given a very interesting account of mix design development in Reference 2.

Tar was used as a binder in the first bituminous pavements placed mostly in Washington, D.C. between 1868 and 1873. The importance of aggregate proportioning was not understood at that time, and the mixing process was not mechanized. These tar mixes did not perform well as surface mixes. However, their performance as bases was satisfactory. Tar was also in competition with Trinidad Lake asphalt during that period.

Clifford Richardson, considered as an early great American asphalt technologist, obtained and analyzed the samples of those earlier bituminous bases in 1906. He understood the importance of material selection, especially the significant role of fine aggregate fractions. He studied and documented the important principles of HMA design including the voids in the mineral aggregate (VMA) and air void content. Richardson published a book, "The Modern Asphalt Pavement," in 1905 (3).

The Pat Test

Richardson has described the "Pat Test" in his book. It can probably be considered as the first mix design test to determine the optimum binder content of an HMA mix. It was applied to sheet asphalt (a hot sand asphalt mix) used as surface mix. Pat samples of sheet asphalt containing different amounts of binder were pressed (compacted) against a brown manila paper, and the residual stains on the paper were assessed visually. A heavy stain indicated too much binder, a light stain indicated too little binder, and a medium stain indicated "just right" or an optimum binder content. Richardson provided comparative illustrations of these stains in his book. The Pat Test was still being used in the early 1920s for fine-graded mixes and for the fine aggregate portion of coarse mixes. Richardson recommended a top size of about one inch or larger for the

aggregate in dense graded HMA base mixes for heavy traffic. However, he used the visual assessment to establish the optimum binder content.

Large Stone Mixes

Frederick Warren obtained a patent for the so-called Bitulithic pavement around the turn of the twentieth century. The Bitulithic pavement allowed larger aggregate size up to three inches. Bitulithic pavements containing 3/4 inch (19 mm) and 1¼ inch (32 mm) maximum aggregate sizes have been documented in Rhode Island and Texas, respectively. Larger top size mixes were placed in other states. These large stone mixes performed better than thick sheet asphalt which had a tendency to rut when subjected to traffic consisting of slow-moving carts with narrow, steel-rimmed wheels or solid-tired trucks in the 1900s.

The City of Topeka, Kansas, was sued by Warren Brothers in 1909 for alleged patent infringements. This resulted in the development of the "Topeka decree" of 1910 which, among other factors, stated that the Bitulithic patent would not be infringed when the top size of the aggregate was less than one-half inch. The "Topeka decree" encouraged an extensive use of fine-graded mixes throughout the U.S.A. These mixes were easier to shovel and rake than the large stone mixes, provided smoother surfaces, and were more suitable for mechanized paving operations. Fortunately, these fine-grained mixes benefited from the development and usage of pneumatic tires on heavy vehicles. Pneumatic tires had significantly lower contact pressures than solid tires and steel-rimmed wheels. Although the Bitulithic patent expired in 1920, the fine-grained mixes continued to be used extensively and performed reasonably well. However, in the 1960's through the 1980's tire designs had changed, tire pressures had increased significantly, and traffic load and volume had also increased substantially. This led to some premature rutting of heavy duty pavements and a trend to return to the use of large stone mixes to minimize rutting.

However, in the 1990s, Superpave was adopted and the mixes were improved primarily through the use of more angular aggregate and more

modified asphalts. These changes resulted in improved performance of asphalt mixtures. There has been less tendency to use large stone mixtures due in part to the problems with segregation, permeability, and difficulty in obtaining a tight dense surface.

Dense Gradation

Roy Green, an associate professor at the Agricultural and Mechanical College of Texas, now Texas A&M University, studied several Bitulithic pavements in Texas, especially the fine aggregate fractions of Bitulithic mixes. He developed a procedure for obtaining an ideal dense gradation for these mixes. He simply connected the percent passing the No. 200 (0.075 mm) sieve with 100 percent passing top size aggregate sieve on a gradation chart, and plotted the intermediate sieve positions on this straight line. This procedure facilitated comparison of gradations of different mixes. Green developed this method 42 years before Goode and Lufsey developed their 0.45 power gradation chart used to identify the maximum density line.

Hubbard-Field Method

A test method for determining the optimum asphalt content of sheet asphalt surfaces and sand asphalt bases was devised by Hubbard and Field in the middle 1920s. The test consisted of determining the maximum load developed as a specimen 2 inches (50.8 mm) in diameter by 1 inch (25.4 mm) high was forced through a 1.75-inch (44.4 mm) diameter standard orifice. The load was reported as the stability value. Stability numbers corresponding to various asphalt contents were plotted and the optimum binder content determined. This was probably the first attempt to quantify empirical mix stability values at various asphalt contents. The method was widely accepted and used in some states for many years. The test was modified in the mid-1950s to test 6-inch (150 mm) diameter specimens to accommodate mixes containing coarse aggregate up to 3/4 inch (19 mm) maximum size. However, the modified

version was used for a very short time because the Marshall test had started to gain popularity during that period.

Hveem Mix Design Method

Reference *4* contains a detailed account of the evolution of the Hveem method. As a resident engineer in California, Francis Hveem began to work with "oil mixes" during the late 1920s. Oil mixes, which were a combination of fairly good quality gravel and slow-curing liquid asphalts, were being used in California to obtain an intermediate type surfacing for moderate traffic conditions. When mechanical pavers were introduced in 1937, it became possible to place mixes with heavier grades of asphalt cements. Hveem noticed that there was a relationship between the gradation of the mineral aggregate and the amount of oil required to maintain a consistent color and appearance of the mix. He subsequently found a method for calculating surface area of aggregate gradations developed by a Canadian engineer, L.N. Edwards. Refinements in the method occurred as Hveem realized that the film thickness of binder on a particle decreased as the diameter of the particle decreased. The kerosene equivalent test was developed to take into account differences in oil requirements as the absorption and surface roughness of aggregates varied.

Hveem realized that having the proper oil content did not guarantee good performance relative to rutting; therefore, another test was needed to evaluate the stability, or the ability to resist the shear forces applied by wheel loads. This led to the development of the Hveem stabilometer (ASTM D1560). In this test, a specimen of 4-inch (101.6 mm) diameter and 2½-inch (63.5 mm) height is subjected to a vertical load on the flat surface of the sample and the amount of vertical load that is transmitted horizontally is measured. The circumferential perimeter of the specimen is restrained by a neoprene diaphragm and is surrounded by an oil reservoir to simulate field loading conditions. Empirical stability numbers are obtained at various asphalt contents. Specimens are prepared with a kneading compactor, which is a hydraulic device that

applies pressure to the specimen through a hydraulically operated tamper foot. The foot is raised after a specified pressure is sustained by the specimen, the base rotates 1/6 of a revolution, the tamper foot is automatically lowered, and the specified pressure is applied again around the perimeter of the specimen. The area of the tamper foot is one-fourth the cross sectional area of the specimen.

A second mechanical test device called a cohesiometer was developed along with the Hveem stabilometer. It was designed to measure the cohesive strength across the diameter of a compacted specimen on which the stability test had already been conducted. The specimen is placed in the cohesiometer, the specimen is secured, and the load is applied by lead shots flowing from a reservoir into a bucket at the end of the movement arm (details are given in ASTM D1560). Load is applied until the specimen fails, and at that time the shot supply is automatically shut off. The total applied force is determined and the cohesion calculated by a formula. This test was aimed at measuring a tensile property of the oil mixes that could be related to a minimum level to preclude raveling of surface mixes under horizontal surface forces. This test proved to be of little value in characterizing HMA surfacing. HMA surfacings were made with asphalt cement and always had cohesion values large enough to prevent raveling. Therefore, when oil mixes were replaced by HMA surfacings after World War II, the cohesiometer test served no real purpose and it gradually fell out of favor.

By 1959 the Hveem stabilometer procedure for design of HMA mixes had evolved into its final form. Several states, primarily in the west, adopted the Hveem procedure. However, many states modified various elements in the procedure.

Marshall Mix Design Method

A detailed account of the development of the Marshall method is given in References 5 and 6.

The earliest version of the Marshall mix design method was developed at the Mississippi Highway Department by Bruce Marshall around 1939. The Corps of Engineers Waterways Experiment Station (WES) began a study in 1943 to develop a simple portable apparatus for designing asphalt mixtures for airfield pavements. This was primarily prompted by the increase in aircraft wheel loads during World War II. Tire pressures and, therefore, contact pressures also increased, warranting a suitable mix design method to cope with increased tire pressures. The Corps began experimenting with Bruce Marshall's apparatus and embarked on a series of laboratory and field experiments. Mixtures were designed in the laboratory using a variety of compactive efforts in an attempt to produce densities in the laboratory which were similar to those achieved in the field under construction and simulated aircraft loadings. The laboratory compactive efforts involved different drop hammer weights; different combinations of numbers of blows per side and/or compressive loads; different compactor foot designs including both size and shape; and different mold base shapes and materials.

One goal of the laboratory compaction study was to adopt a sample preparation procedure that would involve minimum effort and time but would provide a basis for selecting the proper optimum asphalt content. It was also desirable to develop a method that was portable and which could be taken to the field for quality control purposes. Based on the results of this study, a 10-pound (4.54 kg) hammer with a 3 7/8-inch (98.4 mm) diameter foot was selected, and a compaction effort of 50 blows per side of the specimen was adopted as a standard.

Test sections trafficked with loads and tire pressures of 15,000 lbs (50 psi), 37,000 lbs (110 psi), and 60,000 lbs (110 psi) [66.7 kN (345 kPa), 164.4 kN (759 kPa), and 266.7 kN (759 kPa)] for up to 3500 coverages showed no significant increase in density after traffic, regardless of wheel load. Using the results from tests conducted on field-secured specimens, the limits shown in Table 4-1 were established as the criteria for mixture design:

Table 4-1. 50 Blow Marshall Criteria for Surface Mixes (5)

Test Property	Asphalt Content Selection		Limiting Criteria	
	HMA	Sand Asphalt	HMA	Sand Asphalt
Stability, lbs. (N)	Peak	Peak	500 min	500 min
Flow, 0.01 inch (0.25 mm)	----	----	20 max	20 max
Unit Weight Total Mix, pcf (g/ml)	Peak	Peak	----	----
Voids Total Mix, %	4	6	3- 5	5- 7
Voids Filled with Asphalt, %	80	70	75-85	65-75

During the late 1940s and early 1950s, aircraft size and weight continued to increase resulting in tire pressures increasing to 200 psi (1380 kPa) in order to keep aircraft gear and tire sizes about the same. As a result the laboratory compactive effort was increased to 75 blows per side of specimen based on additional test sections that were constructed and tested.

In addition to the development of the high tire pressure criteria in the 1950s, WES noted performance problems, especially rutting, on a number of projects. Investigations revealed that large quantities of natural sand were present in those mixes experiencing rutting problems. Attempts to control the use of natural sands resulted in raising the Marshall stability to 1800 lb (8000 N) and subsequently a limitation on the allowable amount of natural sand that could be used in airfield pavements. The initial limit on natural sand was 10 percent of the total aggregate, but this was later raised to 15 percent.

The volumetrics originally specified for airfields were later changed. The required air voids was later changed to 3-5 percent. The voids filled with asphalt requirements were later changed to 70-80 percent. Also voids in mineral requirements (VMA) were later included in the requirements.

In reviewing the development of the Marshall method, it is apparent that a controlling factor is the relationship between the density achieved in the field under traffic and that produced in the laboratory with the specified compactive effort. When field conditions changed due to changes in traffic, WES monitored the field conditions and adjusted the laboratory procedures to produce densities similar to those was changes in the field density brought about by changes in traffic.

Applying this principle of empirical design to highway practice, one sees the importance of determining field conditions induced by traffic before making changes in the laboratory procedure. Only by knowing field conditions can proper adjustments be made in the laboratory since the laboratory conditions must replicate field conditions.

In a survey performed in 1984 (*1*), it was shown that 38 states used some version of the Marshall method while 10 states used the Hveem method. By 2009, almost all states had converted over to the Superpave mix design method.

Superpave Mix Design Method

A detailed description of the original Superpave mix design method is provided in Reference *7*. There have been a number of changes as Superpave was adopted by the state DOTs and as more experience was gained (*8, 9, 10, 11, 12, 13*).

The Superpave mix design method is a direct result of the Strategic Highway Research Program that was initiated in the late 1980's. The primary purpose of this program was to develop a rational mix design procedure that could be used to better predict performance of HMA. A number of compaction methods that had been used to compact HMA samples were investigated but gyratory compaction was selected because it oriented the aggregate particles in a way more similar to that observed under traffic. Several gyratory compactors were investigated to be used to compact samples for this design method but in the end a new gyratory compactor was developed. With this gyratory compactor, the angle of

gyration and the vertical pressure are fixed and the number of gyrations is varied to simulate different traffic levels.

The size of sample was selected to be 6 inches in diameter compared to 4 inches for the Marshall samples. The larger sample size was believed to be needed to allow larger maximum aggregate size to be used. With the Marshall method the aggregate has to be one inch or smaller but with the Superpave method the aggregate can be as large as 1.5 inches without causing significant problems.

Even though Superpave is a mix design procedure it also includes requirements for aggregates and asphalt binder. Originally there was a restricted zone as part of the specification requirements but the restricted zone is no longer part of the design procedure. The material requirements that were made a part of the Superpave mix design system require higher quality aggregate than most existing specifications thus helping to ensure a higher quality mixture.

The Superpave mix design method uses volumetrics to select the optimum asphalt content. Mixtures are compacted to provide a density in the laboratory equal to the estimated density in the field after various levels of traffic. The number of gyrations is varied to simulate anticipated traffic. Initial work developed data to set the number of gyrations for various traffic levels. The data to support this effort was limited so there have been 2 additional more detailed studies to better establish the number of gyrations for a given traffic level.

The air voids at $N_{initial}$, N_{design}, and $N_{maximum}$ are measured to evaluate the mixture quality. The mixture should meet some minimum void criteria at $N_{initial}$, 4 percent air voids at N_{design}, and at least 2 percent air voids at $N_{maximum}$. The number of revolutions for $N_{initial}$, N_{design}, and $N_{maximum}$ are defined later in this chapter. The $N_{initial}$ is used to help ensure that the mix is not tender during construction and to help ensure that rutting does not occur. The $N_{maximum}$ is used to help ensure that rutting does not occur. There has been much debate about the need to use both $N_{initial}$ and $N_{maximum}$ but both are included as part of the Superpave criteria.

During the development of Superpave, aggregate blends were defined as fine graded or coarse graded based on their gradation.

Initially, this refereed to aggregate gradations that were above or below the restricted zone. Those gradations above the restricted zone were referred to as fine graded mixtures and those below the restricted zone were referred to as coarse graded mixtures. After the restricted zone was removed as a requirement for Superpave, coarse or fine graded mixtures generally refer to gradations above or below the maximum density line at the No. 4 (4.75mm) sieve. Those aggregates above the maximum density line at the No. 4 sieve are referred to as fine graded and those below the maximum density line are referred to as coarse graded mixtures.

Initially it was recommended that coarse graded mixtures be used to help ensure that rutting does not occur but many state DOTs have dropped this requirement since coarse graded mixtures often lead to mixtures that are permeable. Some state DOTs now require that fine graded mixtures be used while others do not specify whether fine graded or coarse graded should be used.

In early Superpave gyratory machines it was noticed that machines from different manufacturers often gave varying results even when the equipment met all of the specifications. After time, it was learned that the major reason for this difference was the internal angle of gyration. Even though the angle of gyration was set externally, the internal angle could vary depending on the stiffness of the equipment leading to different densities in samples compacted with machines having different stiffnesses. Research led to devices that could measure the internal angle of gyration and this has helped to solve the problem of different densities between the various gyratory compactors. This mixture design method has been fully implemented by most State DOTs. Initially it was desired to have every state adopt Superpave completely as recommended at the national level. However, most state DOTs have made at least some changes to the nationally recommended procedure to satisfy localized issues. The one thing that is still missing is a nationally recommended performance test(s). Many states have already adopted various performance tests due to lack of a nationally recommended test and as a result it will be very difficult to eventually have one test that will be used

by all states. The most common performance test being used to help control rutting is some type of wheel tracking test.

OBJECTIVES AND ELEMENTS OF MIX DESIGN

Objectives

Regardless of the mix design procedure used, the primary reason for conducting a mix design for hot mix asphalt materials is to determine if a suitable mixture can be prepared with the available materials and to determine the best combination of these materials for optimum performance. During mix production, the mix design will need to be revised to meet field conditions since the materials will change some from the mix design to plant production. Also it is impossible to obtain truly representative samples during the mix design process resulting in a mix design that will be difficult to duplicate during field production. The items addressed below must be evaluated during the mix design to ensure that optimum performance is obtained.

Resistance to Permanent Deformation. Generally to provide for the longest life, the mix design should provide for as much asphalt as possible in the mixture as long as excessive permanent deformation does not occur. The mix should not distort or displace excessively when subjected to traffic. Resistance to permanent deformation (or rutting) becomes critical at elevated temperatures during hot summer months when the viscosity of the asphalt cement binder is low and the traffic load is primarily carried by the mineral aggregate structure. Resistance to permanent deformation is controlled by selecting quality aggregates with proper gradation and selecting the asphalt content so that adequate voids exist in the mix. The proper grade of asphalt binder must also be selected to ensure that excessive rutting does not occur during the hot weather months. Test methods such as static or dynamic creep or wheel tracking tests discussed in Chapter 5 may be more suitable for predicting

permanent deformation. Generally mixtures with lower total asphalt content will have more resistance to permanent deformation.

Fatigue Resistance. The mix should not crack when subjected to repeated loads over a period of time. Complex repeated load (either constant stress or constant strain controlled) tests discussed in Chapter 5 can be run to estimate the number of cycles to cause failure (cracking) in HMA mixtures. Fatigue is typically a pavement design issue and is not normally controlled in the mix design process. However, there are things that can be done during mix design to help minimize fatigue problems. For example, in most cases a mixture with higher asphalt content will have more resistance to fatigue damage than a mixture with lower asphalt content.

Resistance to Low Temperature Cracking. This mix property is important in cold regions which experience low ambient temperatures. As discussed in Chapter 2, low temperature cracking of HMA pavement is primarily influenced by the low temperature properties of the asphalt cement binder. Therefore, a proper selection of binder will minimize this problem. Also a mixture with low air voids, larger asphalt film thicknesses, etc. will be more resistant to aging and low temperature cracking.

Durability. The mix must contain sufficient asphalt cement to ensure an adequate film thickness around the aggregate particles, thus minimizing asphalt cement hardening or aging during production and in service. The compacted mix should not have very high air voids (increased permeability) which will also accelerate the aging process. Generally mixtures with higher asphalt contents will have better durability.

Resistance to Moisture Induced Damage. Some HMA mixes, when subjected to moisture or water, lose adhesion between the aggregate surface and asphalt cement binder. Aggregate properties are

primarily responsible for this phenomenon, although some asphalt cements are more prone to moisture damage (stripping) than others. If an HMA mix is prone to stripping, then antistripping agents should be used. Making the mix impermeable to water also minimizes the problem. Moisture susceptibility of HMA mixes is discussed later.

Skid Resistance. This requirement is only applicable to surface mixes which must be designed to provide sufficient resistance to sliding to permit normal turning and braking movements to occur. Aggregate characteristics such as texture, shape, size, and resistance to polish are primarily responsible for skid resistance. However, the mix should also not contain so much asphalt cement binder that bleeding may occur creating a slippery surface. Skid resistance is discussed in detail in Chapter 8. It is also discussed briefly later in this chapter.

Workability. The mix must be capable of being placed and compacted with reasonable effort. No test method is currently available to quantify workability during the laboratory mix design phase. Workability problems are most frequently discovered during the paving operations. However, suitable adjustments to the mix design can be made quickly to overcome the workability problems. Generally finer mixtures with relatively high asphalt content have better workability than coarser mixtures and mixtures with low asphalt contents.

Gradation and Maximum Density Curves. Blending of aggregates is discussed in Chapter 3. Grading curves are very helpful in making necessary adjustments to the mix design properties such as VMA (voids in the mineral aggregate). The Bailey method has been used by many as a tool to help evaluate the effect of gradation on volumetrics as well as compactibility. Using this method makes it easier during mix design to establish a gradation that will satisfy requirements for VMA. This method is not discussed here but can be evaluated in reference *14*.

Maximum density curves, as mentioned in Chapter 3, are particularly suitable in making comparisons with the grading that is considered for

use in the mix design process. In 1962 Goode and Lufsey proposed that aggregates should be graded using a mathematical concept of packing the void space between aggregates of large diameter with aggregates of smaller diameter (*15*). Nijboer cites the work of Andreasen in determining the grain size distribution of spherical particles of all sizes to produce the most efficient arrangement of packing in a confined space (*16*). The mathematical relationship describing the maximum density gradation to produce the densest packing of particles based on Fuller's curve was given in Chapter 3. However, Goode and Lufsey (*15*) noted that if the gradation corresponding to the exponent of 0.5 is used as proposed by Fuller, then the VMA may be too low to ensure both sufficient air void content and sufficient asphalt cement for durability without encountering field stability problems. Therefore, recommendations in the FHWA Advisory (*17*) include the suggested use of the 0.45 power curve to evaluate and adjust aggregate gradations. The most convenient way to evaluate gradation is to plot the combined gradation on a plot on which the FHWA 0.45 power gradation line has been graphed such as shown in Figure 4-1.

There has been some confusion on how the various agencies prepare the plots representing the 0.45 power curve. In referring to the original Goode and Lufsey (*15*) paper, it is not clear in the paper how the maximum particle size D or the minimum particle size is determined and plotted. There appear to be three prevalent methods for plotting the maximum density lines.

The solid line in Figure 4-2 shows the gradation of a binder course mix for the following gradation above the ½ inch sieve.

Sieve Size	Percent Passing
1½ inch (37.5 mm)	100
1 inch (25 mm)	90
½ inch (12.5 mm)	61

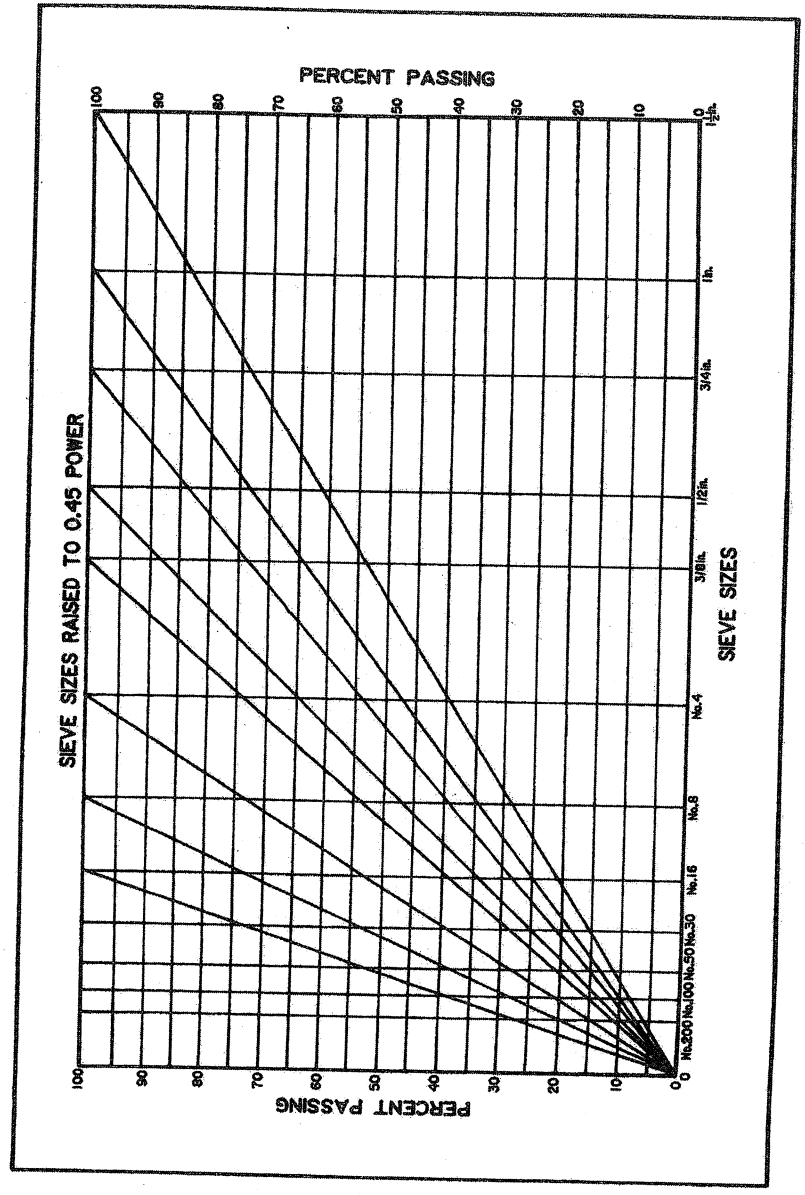


Figure 4-1. Recommended FHWA Maximum Density Curves Based on the 0.45 Power Gradation Chart

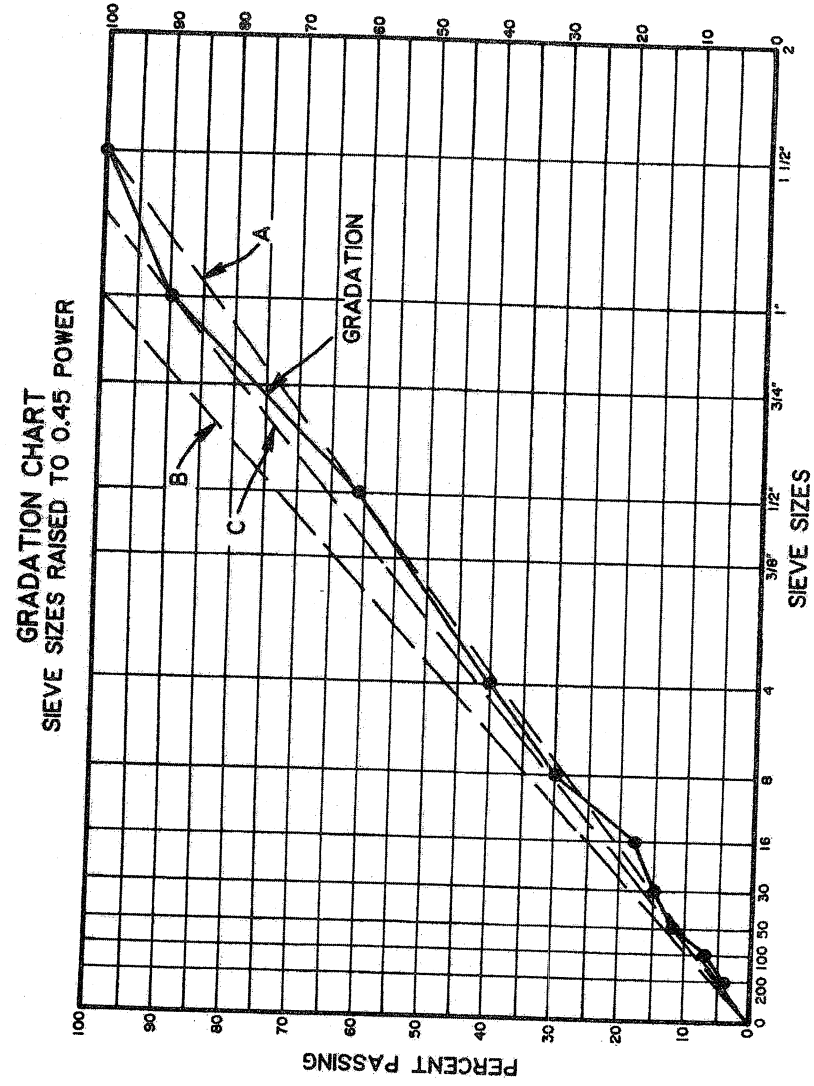


Figure 4-2. Three Common Methods of Plotting Maximum Density Lines on 0.45 Power Gradation Chart

These gradation data are used three different ways to produce the maximum density lines labeled A, B, and C in Figure 4-2. The construction of each line is described below:

Maximum Density Line "A" - obtained by drawing a straight line from the origin at the lower left of the chart to the maximum (top) particle size (1½ inch (37.5 mm) in the example) at the top.

Maximum Density Line "B" - obtained by drawing a straight line from the origin to the *nominal* maximum particle size (1 inch (25 mm) in the example) at the top. The *nominal* maximum particle size is defined as one sieve larger than the first sieve size that has less than 90 percent passing. So the first sieve to have less than 90 percent passing is the ½ inch sieve, so the nominal maximum aggregate size is one size larger or 1 inch in the example.

Maximum Density Line "C" - obtained by drawing a straight line from the origin to the percentage point plotted for the largest sieve with material retained (90 percent passing point for 1 inch (25 mm) sieve in the example).

The Maximum Density Line "C" was recommended by Goode and Lufsey of the then Bureau of Public Roads (now the FHWA) who developed this gradation chart in the early 1960s. Their preference is stated in the discussion of the paper which was reported in the Proceedings of the Association of Asphalt Paving Technologists (Volume 31, 1962). Line "C" is considered to be the most realistic maximum density line. It is very important to have consensus among designers on this issue, especially since some state specifications require that the mix gradation be below the maximum density line. The 0.45 power curve for maximum density line should be used only as a guide and should *not* be incorporated into specifications that require that all gradations be on, above, or below this line. Satisfactory mixes can be produced under all three conditions. However, a gradation in close proximity to the maximum density line generally gives low VMA, resulting in a less durable mix that is more sensitive to slight variations in asphalt content during production. To avoid these difficulties gradations are usually shifted away from the maximum density line.

The following general recommendations should be considered in mixture design:

1. If the fine aggregate is predominately round natural sand, it is usually preferred to stay below the maximum density line to minimize the amount and the effect of the sand.
2. If crushed or very angular fine aggregate is used, the gradation may be above the maximum density line. However, these mixes may not be economical since the voids must be filled with asphalt cement in order to meet the air voids criteria in mix design. These finer mixtures typically require a higher asphalt content.
3. It is undesirable to have a hump in the gradation in the No. 30-50 sieve size range, but this should especially be avoided when natural sands predominate in the fine aggregate. Humps in this size range tend to produce mixes which are tender, i.e., they exhibit high plastic distortion under normal compaction equipment immediately after laydown.
4. Mixtures below the maximum density line must be well compacted otherwise the mixtures will tend to be permeable. For this reason some mix designers now design mixtures on the fine side of the maximum density line.

These preceding recommendations should be considered especially when the mixture being placed is giving problems during laydown or compaction or experiencing appreciable performance problems during service.

Weight Volume Relationships

A basic understanding of weight-volume relationships of compacted HMA is important to understanding mixture performance and some factors that affect performance. This is very important to understand during conduct of mix design and quality control/quality assurance in the

field. Mix design is based on the volumetric properties of an asphalt mixture and the primary purpose of mix design is to determine the proportion of asphalt cement and aggregates required to produce a mixture with the desired properties. It is the volume of the components of an asphalt mixture that really affect performance and not the weight so the component volumes must be determined. Measurements of the volume of aggregates and asphalt cement in the laboratory or in the field are very difficult and not practical. Therefore, to simplify the measurement problem, weights are used instead of volume, and the specific gravity of the components and the mixture is used to convert from weight to volume. Three very important parameters in controlling the quality of the designed mixture are percent of air voids in total mix (VTM), percent of voids in the mineral aggregates (VMA), and percent of the voids filled with asphalt. Figure 4-3 illustrates these volumes in a simplistic format. If we assume that we can separate the various components in a mixture, we can show the various components as illustrated. The mixture will contain air voids (V_v), effective asphalt content (V_{EAC} and W_{EAC}) which is amount of asphalt that is not absorbed into the aggregate, absorbed asphalt content (V_{AAC} and W_{AAC}) which is the amount of the total asphalt that is absorbed into the aggregate, entrapped air (V_{EA}) which is the amount of the external voids in the aggregate that is not filled with asphalt, and aggregate (V_{AGG} and W_{AGG}). The total weight and total volume is referred to as W_T and V_T.

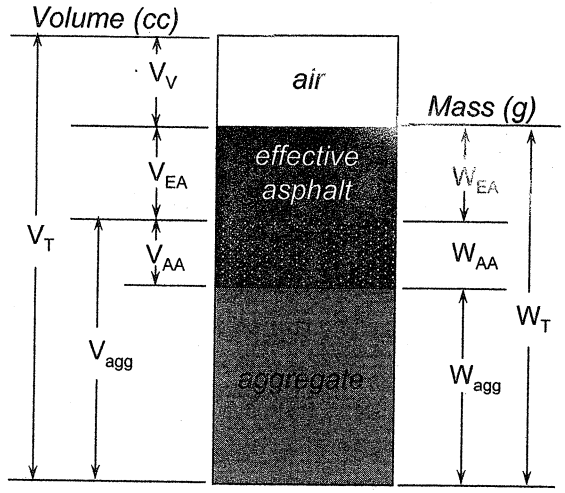
Aggregate specific gravity is a very important property but it must be clear which type of specific gravity is being used. The weight of a sample of aggregate is easy to determine but the volume is more difficult. There are 3 aggregate volumes that might be used depending on which specific gravity is being determined. The three types of specific gravity include: apparent, bulk, and effective. One of the aggregate volumes consists of the solid aggregate particle without the external voids in the aggregate. Recall that the external voids in the aggregate are determined in the specific gravity test when the apparent and bulk specific gravity values are determined. The volume of the moisture in the SSD condition of the aggregate is equal to the external

voids in the aggregate. So for a given weight of aggregate the three possible volumes of the aggregate are the volume represented by the apparent specific gravity (mineral aggregate not including external voids), the volume represented by the bulk specific gravity of the aggregate (mineral aggregate plus external voids), and the volume represented by the effective specific gravity of the aggregate (mineral aggregate plus external voids minus volume of absorbed asphalt).

If there is no absorption of asphalt into the aggregate then the effective specific gravity of the aggregate will be equal to the bulk specific gravity of the aggregate. If the absorption of asphalt fills the external voids then the effective specific gravity will be equal to the apparent specific gravity. A good estimation is that the effective specific gravity of the aggregate is one-half the way between the apparent and bulk specific gravities. When the mixture is heated to a higher temperature then or kept hot for a prolonged amount of time, there is more absorption of asphalt into the aggregate resulting in a higher measured effective specific gravity of the aggregate. When the aggregate absorption is small the effective specific gravity of the aggregate will not be significantly affected by temperature or length of time the mix is kept hot.

Definitions. A more complete understanding of weight-volume relationships start with definitions. The definitions given below are consistent with those advanced by ASTM, AASHTO, Asphalt Institute, and National Asphalt Pavement Association.

Voids in the Mineral Aggregate (VMA) - The volume of intergranular void space between the aggregate particles of a compacted paving mixture. This volume includes the volume of air voids in between that asphalt coated particles plus the volume of the effective asphalt (that asphalt not absorbed into the aggregates) This can be seen in Figure 4-3 and 4-4.



$$VTM = \frac{V_V}{V_T} \times 100 \qquad AC = \frac{W_{AC}}{W_T} \times 100 = \frac{W_{EA} + W_{AA}}{W_T} \times 100$$

$$VMA = \frac{V_V + V_{EA}}{V_T} \times 100 \qquad VFA = \frac{V_{EA}}{V_{EA} + V_V} \times 100$$

where,

V_T = Total volume of compacted specimen
V_V = Volume of air voids
V_{EA} = Volume of effective asphalt
V_{AA} = Volume of absorbed asphalt
V_{Agg} = Volume of aggregate
W_T = Total weight of compacted specimen
W_{EA} = Weight of effective asphalt
W_{AA} = Weight of absorbed asphalt
W_{Agg} = Weight of aggregate
VTM = Voids in total mix, percent
VMA = Voids in mineral aggregate, percent
VFA = Voids filled with asphalt, percent
AC = Asphalt content by weight of mix, percent
γ_W = Unit weight of water

Figure 4-3. Weight-Volume Relationships for Asphalt Concrete

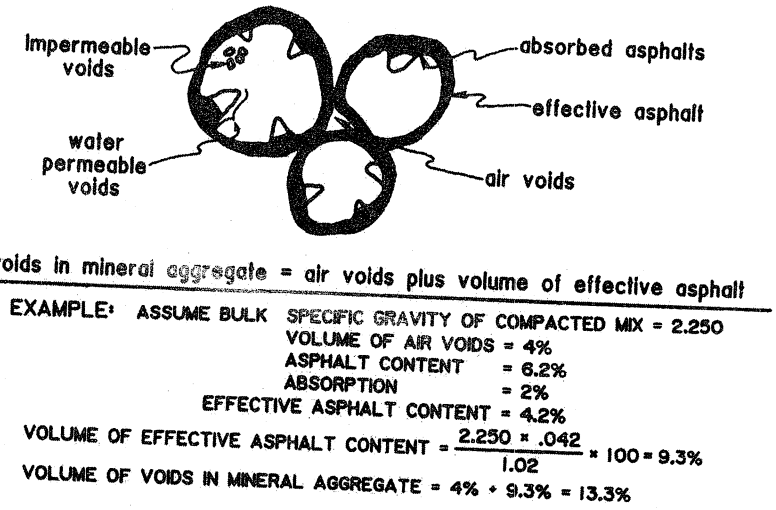

Impermeable voids
absorbed asphalts
effective asphalt
water permeable voids
air voids
voids in mineral aggregate = air voids plus volume of effective asphalt
EXAMPLE: ASSUME BULK SPECIFIC GRAVITY OF COMPACTED MIX = 2.250
VOLUME OF AIR VOIDS = 4%
ASPHALT CONTENT = 6.2%
ABSORPTION = 2%
EFFECTIVE ASPHALT CONTENT = 4.2%
VOLUME OF EFFECTIVE ASPHALT CONTENT = 2.250 × .042 / 1.02 × 100 = 9.3%
VOLUME OF VOIDS IN MINERAL AGGREGATE = 4% + 9.3% = 13.3%

Figure 4-4. Diagrammatic Representation of Air Void and Voids in Mineral Aggregate (VMA) (*18*).

Voids in Total Mix (VTM) - The total volume of the small pockets of air between the coated aggregate particles throughout a compacted paving mixture, expressed as a percent of the bulk volume of the compacted paving mixture. In Figure 4-3 this is shown as the air voids in the mixture (V_v).

Voids Filled with Asphalt Cement (VFA) - The percent of the volume of the VMA that is filled with asphalt cement. In Figure 4-3 this would be equal to the volume of effective asphalt divided by the volume of VMA expressed as a percentage.

In order to calculate VTM, VMA, or VFA, it is necessary to have data on the bulk specific gravity of the compacted HMA (ASTM D1188 or D2726), the appropriate specific gravity values of the individual components of the mixture, the theoretical specific gravity of the HMA (this will be discussed below), and the asphalt content.

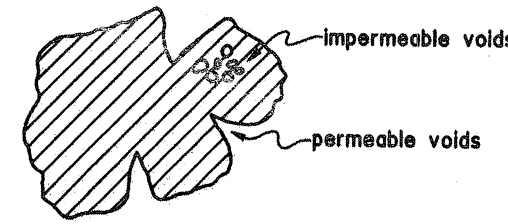
When one looks at a compacted asphalt mixture it consists of the following components (Figure 4-3): aggregate (V_{Agg} and W_{Agg}), air voids

outside coated aggregate particles (V_V), entrapped air between asphalt coating and aggregate particle, effective asphalt (W_{EA} and V_{EA}) and absorbed asphalt (W_{AA} and V_{AA}). The volume of the aggregate in the schematic can be three different values depending on which specific gravity of aggregate is used. For example, the volume of the aggregate can be equal to the volume of the aggregate particle including the external voids in the aggregate (entrapped air within the pores of the aggregate (portion of the pores not filled with asphalt) plus the absorbed asphalt). If the weight of the aggregate along with this volume is used the calculated specific gravity is equal to the bulk specific gravity of the aggregate. A second volume that may be considered is the volume of the aggregate particles excluding the entrapped air and the absorbed asphalt. If the weight of the aggregate and this volume of the aggregate is used the calculated specific gravity of the aggregate is the apparent specific gravity. A third volume is the volume of the aggregate particles excluding the external voids plus the volume of the entrapped air. Using this volume and the weight of the aggregate particles to calculate the specific gravity of the aggregate results in the effective specific gravity of the aggregate.

In order to facilitate further discussions relative to voids, definitions are needed for the methods used for measuring specific gravity of the aggregates and their relevancy to voids considerations. Specific gravity is needed in voids calculations to convert from weight measurements to volume determinations. An understanding of the weight volume relationships is needed. Apparent and bulk specific gravities of the mineral aggregates were discussed in detail in Chapter 3. Their definitions only will be given here. An identification of terms used in the weight volume relationships is provided below:

Apparent Specific Gravity (ASTM C127 and C128) - The ratio of the oven dry weight in air of a unit volume of an impermeable material at a stated temperature to the weight of an equal volume of gas-free distilled water at a stated temperature (see Figure 4-5). Apparent specific gravity is normally used for weight to volume calculations of the mineral filler

only, since bulk specific gravity values of this fraction are very difficult to obtain.



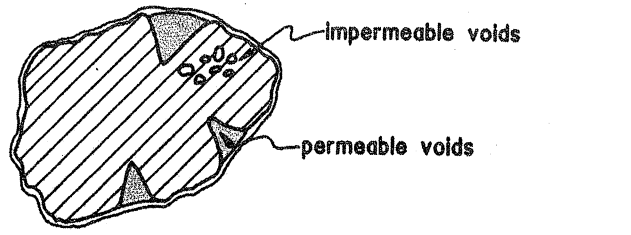
$$\text{apparent specific gravity} = \frac{\text{weight of oven dry aggregate}}{\text{volume of aggregate plus volume of impermeable voids}}$$

VOLUME IS DETERMINED BY DISPLACEMENT IN WATER WITH OVEN DRY AGGREGATE OR USING A SATURATED SURFACE DRY AGGREGATE AND THEN SUBTRACTING THE VOLUME OF WATER EQUAL TO THE AGGREGATE WATER ABSORPTION

Figure 4-5. Diagrammatic Representation of Apparent Specific Gravity (*18*)

Bulk Specific Gravity, Dry (ASTM C127 and C128) - The ratio of the oven dry weight in air of a unit volume of a permeable material (including external voids) at a stated temperature to the weight of an equal volume of gas-free distilled water at a stated temperature (see Figure 4-6).

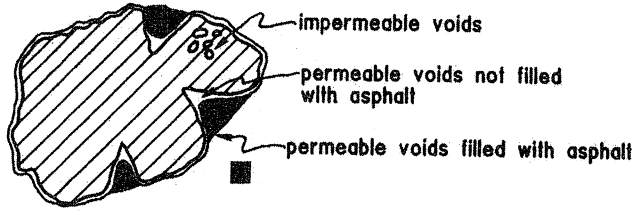
Effective Specific Gravity of an Aggregate - The ratio of the oven dry weight in air of a unit volume of a permeable material (excluding voids permeable to asphalt) at a stated temperature to the weight of an equal volume of gas-free distilled water at a stated temperature (see Figure 4-7).



bulk (dry) specific gravity = weight of oven dry aggregate / volume of aggregate plus volume of impermeable voids and permeable voids

VOLUME IS DETERMINED BY DISPLACEMENT IN WATER WITH SATURATED SURFACE DRY AGGREGATE

Figure 4-6. Diagrammatic Representation of Bulk Specific Gravity (*18*)



effective specific gravity of aggregate = weight of oven dry aggregate / volume of aggregate plus volume of impermeable voids plus volume of water permeable voids less volume of absorbed asphalt

EFFECTIVE SPECIFIC GRAVITY DETERMINED BY PROCEDURES OUTLINED IN ASTM D2041.

BULK IMPREGNATED SPECIFIC GRAVITY OF CORPS OF ENGINEERS USUALLY INDICATES MORE ASPHALT ABSORPTION AND HIGHER EFFECTIVE SPECIFIC GRAVITY.

EFFECTIVE SPECIFIC GRAVITY SHOULD FALL BETWEEN APPARENT AND BULK (DRY) SPECIFIC GRAVITY.

Figure 4-7. Diagrammatic Representation of Effective Specific Gravity of Aggregate (*18*)

Bulk Specific Gravity, Compacted Asphalt Concrete (ASTM D1189 and D2726) - The ratio of the weight in air of a unit volume of a compacted specimen of HMA at a stated temperature to the weight of an equal volume of gas-free distilled water at a stated temperature. The

value is used to determine weight per unit volume of the compacted mixture. The bulk specific gravity (G_{mb}) of a compacted asphalt mixture is equal to:

$$G_{mb} = \frac{W_D}{W_{SSD} - W_{Sub}}$$

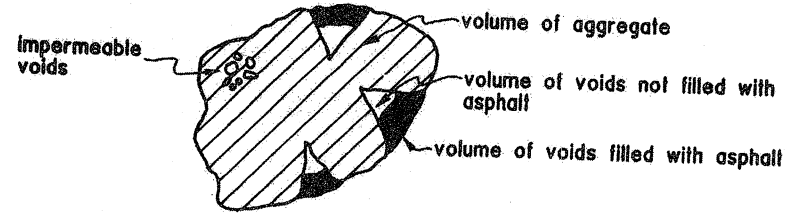
where,

W_D = Dry weight, grams

W_{SSD} = Saturated surface dry weight, grams

W_{Sub} = Saturated surface dry weight submerged in water, grams

Theoretical Maximum Specific Gravity of Bituminous Paving Mixtures (ASTM D2041) - The ratio of the weight in air of a unit volume of an uncompacted bituminous paving mixture at a stated temperature to the weight of an equal volume of gas-free distilled water at a stated temperature (see Figure 4-8). It is also called Rice Specific Gravity (after James Rice who developed the test procedure) or theoretical maximum density (TMD).



maximum theoretical specific gravity of mixture = weight of aggregate plus weight of asphalt / volume of aggregate plus volume of impermeable voids plus volume of permeable voids not filled with asphalt plus total volume of asphalt

$$GMM = \frac{PMM}{\frac{PS}{GSE} + \frac{PB}{GB}}$$

PMM = TOTAL WEIGHT OF MIX
PS = WEIGHT OF AGGREGATE
PB = WEIGHT OF ASPHALT
GSE = EFFECTIVE SPECIFIC GRAVITY OF AGGREGATE
GB = SPECIFIC GRAVITY OF ASPHALT

Figure 4-8. Diagrammatic Representation of Maximum Theoretical Specific Gravity of Mixture (*18*)

A review of the above specific gravity definitions indicates the following:

1. The differences in calculations among apparent, bulk and effective specific gravity is the volume of the aggregate. All three specific gravities use oven dry weights of aggregate.
2. The difference in calculations between the bulk specific gravity of compacted samples and the maximum theoretical specific gravity is the volume since the weights are identical. The volume of the compacted sample includes the air voids within the compacted sample. However, the volume of the theoretical maximum density sample does not include any air voids; therefore this is referred to as the theoretical maximum density.
3. The measured values of the above specific gravities can be conveniently checked for reasonableness by realizing the following:
   (a) The apparent specific gravity will always be equal to or larger than the effective specific gravity which will always be equal to or larger than the bulk specific gravity.
   (b) The maximum theoretical specific gravity will always be equal to or larger than the compacted bulk specific gravity of the HMA mixture.

Calculations. Calculations associated with weight-volume relationships for HMA mixtures can be found in considerable detail in Asphalt Institute publications (*19*). These calculations are summarized below.

Bulk Specific Gravity of Combined Aggregates – HMA may contain aggregate from several sources having different specific gravities. These aggregates are combined to meet the desired gradation requirements. If the aggregate specific gravities are significantly different then they need to be combined for determining weight-volume relationships of the mixtures. The equation used to determine the specific gravity of the combined aggregate was given in Chapter 3.

The bulk specific gravity of mineral filler is difficult to determine accurately. However, apparent specific gravity of the filler can be used and the error is negligible since the amount of filler used is typically small.

Volume of Voids in Mineral Aggregate (VMA) - As noted above, the volume of voids in the mineral aggregate VMA is an important factor for mixture design. The formula for VMA can be developed as follows based on the weight-volume relationships shown in Figure 4-3. It is standard practice to use the bulk specific gravity of aggregate for calculating VMA.

$$VMA = \frac{V_T - V_{Agg}(bulk)}{V_T} 100$$

simplifying,

$$VMA = 100 - \frac{V_{Agg}(bulk)}{V_T}(100)$$

substituting volume with weight divided by specific gravity gives

$$VMA = 100 - \frac{W_{Agg} / G_{sb}}{W_T / G_{mb}} 100$$

substituting,

$$W_{Agg} = W_T - W_T P_b$$

and simplifying

$$VMA = 100\left(1 - \frac{G_{mb}(1 - Pb)}{G_{sb}}\right) \qquad (4\text{-}1)$$

where,

V_{Agg} = Volume of aggregate
V_T = Total volume of compacted mixture
W_{Agg} = Weight of aggregate
W_T = Total weight of mixture

G_{sb} = Bulk specific gravity of aggregate
G_{mb} = Bulk specific gravity of compacted mixture
P_b = Asphalt content

Air Voids in Compacted Mixture (VTM) - The formula for calculating the percent air voids can be developed as follows:
By definition,

$$VTM = \frac{V_V}{V_T} \times 100$$

substituting,

$$V_V = V_T - V_{EA} - V_{Agg}$$

$$VTM = \left(\frac{V_T - V_{EA} - V_{Agg}}{V_T} \right) 100$$

$$= \left(1 - \frac{V_{EA} - V_{Agg}}{V_T} \right) 100$$

multiply the numerator and denominator by W_T and simplify,

$$VTM = \left(1 - \frac{W_T / V_T}{W_T / (V_{EA} + V_{Agg})} \right) 100$$

substituting,

$$VTM = \left(1 - \frac{G_{mb}}{G_{mm}} \right) 100 \qquad (4\text{-}2)$$

where,

VTM = Voids in total mix
V_V = Volume of air voids
V_T = Total volume of compacted specimen
V_{EA} = Volume of effective asphalt cement
V_{Agg} = Volume of aggregate (bulk)
W_T = Total weight of compacted specimen
G_{mb} = Bulk specific gravity of compacted specimen

G_{mm} = Maximum theoretical specific gravity of mixture

Voids Filled with Asphalt (VFA) - VFA is simply defined as the percentage of VMA filled with asphalt cement. The following formula is used to calculate the VFA:

$$VFA = \frac{VMA - VTM}{VMA} \times 100$$

Effective Specific Gravity of Aggregate - The effective specific gravity (G_{se}) is back calculated from the maximum specific gravity (G_{mm}) of the mixture. Tests for G_{mm} are conducted in a way to determine the voidless volume of the loose mixture. The formula for G_{se} can be derived as follows:

$$G_{se} = \frac{W_{Agg}}{V_{Eff}}$$

The effective specific gravity (G_{se}) of the aggregate is equal to the weight of aggregate divided by the effective volume of the aggregate (V_{Eff}). The effective volume is the apparent volume of the aggregate plus the external voids minus the amount of external voids filled with asphalt cement. In the test for maximum specific gravity (G_{mm}), the total volume of the voidless mix is measured and the volume of asphalt cement is calculated from the weight of asphalt cement and the specific gravity of asphalt cement. The effective volume of the aggregate is determined by subtracting the volume of asphalt cement from the total volume of the mixture. Hence,

$$G_{se} = \frac{W_T - P_b (W_T)}{V_T - V_{AC}}$$

substituting

$$G_{se} = \frac{W_T - P_b W_T}{\frac{W_T}{G_{mm}} - \frac{W_{AC}}{G_b}}$$

simplifying

$$G_{se} = \frac{1 - P_b}{\frac{1}{G_{mm}} - \frac{P_b}{G_b}} \qquad (4\text{-}3)$$

where,

W_{Agg} = Weight of aggregate
V_{Eff} = Effective volume of aggregate
V_{AC} = Volume of total asphalt cement
W_T = Total weight of mixture
V_T = Total volume of voidless mixture
P_b = Asphalt content
W_{AC} = Weight of total asphalt cement
G_b = Specific gravity of asphalt cement
G_{mm} = Maximum specific gravity of mixture

Maximum Specific Gravity of Mixtures with Different Asphalt Contents – Normally during mix design, the maximum specific gravity of the HMA mix is measured at one or two asphalt contents (preferably near the optimum asphalt content). If the effective specific gravity of an aggregate is obtained by Equation 3 using the maximum specific gravity of the HMA mix of that asphalt content, the maximum specific gravity of HMA mixes at other asphalt contents can be calculated as shown below. Note that the theoretical maximum specific gravity is needed to calculate the VTM for each asphalt content. Using this process of calculating the theoretical specific gravity is much quicker and easier than having to conduct the test at each asphalt content being evaluated.

$$G_{mm} = \frac{W_{Agg} + W_{AC}}{V_{Eff} + V_{AC}}$$

Substituting

$$G_{mm} = \frac{W_T}{\frac{W_{Agg}}{G_{se}} + \frac{W_{AC}}{G_b}} = \frac{W_T}{\frac{W_T(1-P_b)}{G_{se}} + \frac{W_T P_b}{G_b}}$$

simplifying

$$G_{mm} = \frac{1}{\frac{1-P_b}{G_{se}} + \frac{P_b}{G_b}} \qquad (4\text{-}4)$$

where,

W_{Agg} = Weight of aggregate
W_{AC} = Weight of total asphalt cement
V_{Eff} = Effective volume of aggregate
V_{AC} = Volume of asphalt cement
W_T = Total weight of mix
G_{se} = Effective specific gravity of aggregate
G_b = Specific gravity of asphalt
P_b = Asphalt content, percent by weight of mix

Asphalt Absorption - The percentage of asphalt absorbed by the mineral aggregate is usually expressed as a percentage of aggregate weight rather than as a percentage of weight of total mix. The equation to calculate asphalt absorption can be derived as follows. The equation for absorbed asphalt is

$$P_{ba} = \left(\frac{W_{AA}}{W_{Agg}}\right) 100$$

substituting, weight = volume x specific gravity

$$P_{ba} = \left(\frac{V_{AA}(G_b)}{W_{Agg}}\right) 100$$

The volume of absorbed asphalt is the difference between the bulk volume of the aggregate and the effective volume. Hence

$$P_{ba} = 100\,\frac{(V_{bulk} - V_{Eff})\,G_b}{W_{Agg}}$$

Substituting, volume = weight / specific gravity

$$P_{ba} = 100\,\frac{\left(\frac{W_{Agg}}{G_{sb}} - \frac{W_{Agg}}{G_{se}}\right)G_b}{W_{Agg}}$$

simplifying,

$$P_{ba} = 100\,\frac{G_{se} - G_{sb}}{G_{sb}G_{se}}G_b$$

where

P_{ba}	=	Absorbed asphalt by weight of aggregate
W_{AA}	=	Weight of absorbed asphalt
W_{Agg}	=	Weight of aggregate
V_{AA}	=	Volume of absorbed asphalt
V_{bulk}	=	Bulk volume of aggregate
V_{Eff}	=	Effective volume of aggregate
G_b	=	Specific gravity of asphalt
G_{se}	=	Effective specific gravity of aggregate
G_{sb}	=	Bulk specific gravity of aggregate

Asphalt Film Thickness

Consideration of film thickness is not a direct part of Marshall or Superpave mixture design. The film thickness is controlled indirectly by controlling the gradation (surface area) and the minimum VMA (minimum asphalt content). However, in analyzing mixtures which have exhibited premature distress, it may be desirable to check film thickness.

Therefore, a procedure for calculating film thickness is included in this section.

The thickness of the asphalt cement film around a particular aggregate is a function of the diameter of the aggregate, absorption of the aggregate, and the percent asphalt cement in the mixture. Hveem found that his initial assumption, that each aggregate particle needed to be covered with the same optimum film thickness, was wrong. Rather, Hveem realized that the desired film thickness on a particle decreases as the diameter of the particle decreases. The current technique for calculating film thickness is based on the surface area factors mentioned in Chapter 3. The asphalt cement film thickness is calculated using the following formula:

$$T_F = \frac{V_{asp}}{SA \times W}(1{,}000)$$

where,

T_F	=	Average film thickness, microns;
V_{asp}	=	Effective volume of asphalt cement (liters), (total volume minus absorbed volume);
SA	=	Surface area of the aggregate (m^2 per kg of aggregate); and
W	=	Weight of aggregate (kg).

It should be noted that the volume of asphalt cement V_{asp} is the amount of material available for coating aggregates and, therefore, must have the volume of asphalt cement absorbed by the aggregate subtracted from the total asphalt cement added to the mix. The calculated film thickness is an average film thickness which has been generally correlated with durability. If the asphalt cement film is too thin, air which enters the compacted HMA can more rapidly oxidize these thin films, causing the HMA to become brittle and to fail prematurely by cracking. Additionally, if the aggregates are susceptible to water damage, thin films are more easily and rapidly penetrated by water than thick films

producing the typical manifestations of water damage: rutting, shoving, raveling, and bleeding.

Example 4-1. The surface area of an aggregate has been determined, as discussed in Chapter 3, to be 35.2 square feet per pound (7.23 m^2/kg). The bulk specific gravity of the aggregate was measured to be 2.680 and the effective specific gravity was determined from the theoretical maximum density to be 2.700. Determine the asphalt film thickness in microns if the asphalt cement specific gravity is 1.030. The asphalt content is 5.0 percent and the bulk density of the mixture is 146 pcf (2.340 g/ml).

Recall that

$$T_F = \frac{V_{asp}}{SA \times W}(1{,}000)$$

The effective volume of asphalt cement is equal to the total volume of asphalt cement minus the absorbed volume. For a one cubic foot sample of asphalt mixture the

$$Total\ volume\ of\ asphalt\ cement = \frac{total\ weight \times asphalt\ content}{Sp\ gr \times \gamma_w}$$

$$= \frac{2340g \times 0.05}{1.03(1)} = 113.6\,ml$$

$$Asphalt\ absorbed,\ P_b(by\ wt\ of\ agg) = 100 \times \frac{G_{se} - G_{sb}}{G_{sb}G_{se}}G_b$$

$$= 100\frac{2.700 - 2.680}{(2.680) \times (2.700)}1.030$$

$$= 0.285\%$$

$$= 0.00285$$

$$Weight\ of\ absorbed\ asphalt = 0.00285\ (weight\ of\ aggregates)$$

$$= 0.00285\ (2340 \times 0.95) = 6.33\,grams$$

$$Volume\ of\ absorbed\ asphalt = \frac{6.33}{1.03(1)} = 6.15\,ml$$

$$Effective\ volume\ of\ asphalt = 113.6 - 6.2 = 107.4\,ml$$

Now substituting into equation for film thickness,

$$T_F = \frac{V_{asp}}{SA \times W}(1{,}000)$$

$$= \frac{107.4}{7.23(2340 \times 0.95)}(1{,}000)$$

$$T_F = 6.7\,microns$$

Laboratory Compaction

Based on the discussion of the evolution of both the Marshall and the Hveem methods of mixture design, it is apparent that the laboratory compaction methods included in the initial methods were referenced to field conditions. The reference field conditions were not based on the as constructed sections but on the density achieved after the pavements had sustained a reasonable level of traffic. The Superpave method was also designed to provide a density equal to that after some amount of traffic.

Therefore, it is clear that if the engineer intends to design a mixture in the laboratory that will sustain the loads imposed on it by traffic, that laboratory mixture should be compacted to the same general density (void content) to which the traffic will finally compact it under service

conditions. Otherwise, the traffic levels may be high enough in service to compact the mixture in place to a density significantly higher than that in the laboratory resulting in low in place voids. These low in place voids will likely lead to permanent deformation problems under traffic. Alternatively, the traffic levels may be so light that the compaction energy used in the laboratory produces a density too high for the actual traffic level. The result is too little asphalt cement in the mixture, high in place air voids, and a mixture that will likely experience low durability problems in the field. This second type of mix does not exhibit instability in the field, but the film thickness is too low for good durability under light traffic. It is apparent then that the selection of the laboratory compaction level is critical in designing a mixture that will provide adequate field performance. It is equally important that some type of performance test be used to ensure that satisfactory mechanical properties exist at the designed asphalt content. It is not just a problem of predicting densification, one must also be able to predict mechanical properties of the mixture as it ages.

If surveys show that the laboratory compaction method consistently produces densities substantially lower or higher than those achieved in the field under traffic, the specifying agency should consider changing the laboratory compaction methodology or blows (compaction effort) in order to achieve laboratory densities which are consistent with those being produced by traffic.

MARSHALL MIXTURE DESIGN METHOD

The overall procedure for mixture design always begins with acceptance tests performed on the aggregates and asphalt cement considered for the design. If the acceptance tests on the aggregates and asphalt cement pass then the mix design can be completed using the Marshall procedure. The procedure that follows is appropriate for mixtures using paving grade asphalt cements (graded according to penetration, viscosity, or PG) used with maximum size aggregates not exceeding 1 inch. Since the mold diameter to maximum particle size

ratio should be at least 4, a 1-inch diameter particle is the largest aggregate size permitted in the 4-inch Marshall mold. For situations where mixtures have particle sizes that exceed 1-inch, the modified Marshall method with 6-inch molds should be used. The major steps involved in performing a mixture design begin with source acceptance tests in Steps A and B; specimen preparation in Step C; density and voids analysis in Step D; Marshall stability and flow test in Step E; tabulating and plotting results in Step F; and optimum asphalt content determination in Step G.

STEP A. AGGREGATE EVALUATION

A-1. Determine acceptability of aggregate for use in HMA construction; tests often performed include Los Angeles abrasion, sulfate soundness, sand equivalency, fine aggregate angularity, presence of deleterious substances, polishing, crushed face count for coarse aggregate, and flat and elongated particle count.

A-2. If material is acceptable in Step A-1, then perform other required aggregate tests: gradation, specific gravity, and absorption.

A-3. Perform blending calculations as described in Chapter 3. As a suggested beginning point, plot the specification midband gradation on a FHWA 0.45 power gradation chart and see how close the midband gradation comes to the maximum density line. If it is close to the maximum density line then controlling the VMA is likely to be more difficult. The target gradation should be adjusted to deviate from the maximum density line, especially on the No. 8 (2.36 mm) sieve. VMA increases as the gradation lines move away from the maximum density line plot either up or down. It has generally been observed that mixes finer then the maximum density line have higher VMA values than mixtures coarser than the maximum density line. A good beginning point for estimating percentages of aggregates in a blend can be made by using the target percentage passing the No. 8 (2.36 mm) and No. 200

(0.075 mm) sieves and estimating the quantities of each aggregate required to meet those two percentages. A full gradation check must then be made to check that the percentages passing for other sieve sizes are met. Aggregate blending is discussed in detail in Chapter 3.

A-4. Prepare a specimen weigh-out table by multiplying the percent aggregate retained between sieves times an aggregate total weight of approximately 1150 g, then determine the cumulative weights to produce the target gradation starting with the material passing the No. 200 (0.075 mm) sieve.

STEP B. ASPHALT CEMENT EVALUATION

B-1. Determine appropriate asphalt cement grade for traffic level and climatic area for mixture being designed.

B-2. Verify through testing or certification that the binder meets specification requirements.

B-3. Determine asphalt cement specific gravity and plot viscosity data on a temperature-viscosity plot.

B-4. Determine the ranges of mixing and compaction temperatures from the temperature-viscosity plot:

1. Mixing temperature should be selected to provide a viscosity of 170 ± 20 centistokes.
2. Compaction temperature should be selected to provide a viscosity of 280 ± 30 centistokes.

STEP C. PREPARATION OF MARSHALL SPECIMENS

Prepare Marshall Specimens using the procedures given in Steps C-1 thru C-7 for individual specimens. As an alternate, HMA for a desired asphalt content can be prepared in bulk to produce the required number

of compacted Marshall specimens and to provide a sample for determination of maximum (Rice) specific gravity.

C-1. Dry and sieve aggregates into sizes (preferably individual sizes) and store in clean sealable containers. Separate enough material to make 18 specimens of approximately 1150 g each. Minimum aggregate and asphalt cement requirements to prepare one series of test specimens of a given gradation are 50 lbs. (25 mg) and one gallon (4 liters), respectively.

C-2. Weigh out 18 specimens placing each in a separate container and heat to mixing temperature determined in Step B-4. However, the total aggregate weight should be determined as discussed in Step C-3.

C-3. It is generally desirable to prepare a trial specimen prior to preparing all aggregate batches. Measure height of the trial specimen (h_1) and check against height requirements for Marshall specimens: 2.50 inch (63.5 mm) ± 0.20 inch (5.1 mm). If specimen is outside this range, adjust quantity of aggregate included in a specimen using the following formula:

$$Q = \frac{2.50}{h_1} \times 1150\ g$$

where,

Q = weight of aggregate to produce a 2.5 inches (63.5 mm) tall specimen, g; and

h_1 = height of trial specimen, inches (mm)

C-4. Heat sufficient asphalt cement to prepare a total of 18 specimens. Three compacted specimens each should be prepared at five different asphalt contents. Asphalt contents should be selected at 0.5 percent increments with at least two asphalt contents above "optimum" and at least two below "optimum." See appropriate specifications for a guide on approximate "optimum" asphalt content or the estimate of optimum can be based on experience. Three loose mixture specimens should be

made near the optimum asphalt content to measure Rice specific gravity or theoretical maximum density (TMD). (Note: Some agencies require the Rice specific gravity test at all 5 asphalt contents. However, the precision of the test is best when the mixture is close to the optimum asphalt content. It is preferable to measure the Rice specific gravity in triplicate. After averaging the results from these three tests and calculating the effective specific gravity of the aggregate, the maximum specific gravity for the other four asphalt contents can be calculated using the formula given earlier.)

C-5. Review appropriate specifications to determine number of blows/side and type of compaction equipment required for Marshall specimen compaction.

C-6. Remove hot aggregate, place on a scale, and add the proper weight of asphalt cement to obtain the desired percentage of asphalt cement.

C-7. Mix asphalt cement and aggregate until all the aggregate is coated. It is helpful to work on a heated table. Mixing can be by hand, but a mechanical mixer is preferred.

C-8. Check temperature of freshly mixed material; if it is above the compaction temperature, allow it to cool to compaction temperature; if it is below compaction temperature, discard the material and make a new mix. (Note: Some states permit mixes to be placed back in the oven to be reheated; some states use an oven curing time to better simulate what happens to the HMA mix in the field. This curing time is especially important for aggregates with high absorption since the asphalt absorbed into the aggregate increases with time.)

C-9. Place a paper disc into an assembled, preheated Marshall mold and pour in loose HMA. Check the temperature. Spade the mixture with a heated spatula or trowel 15 times around the perimeter and 10 times over the interior. Remove the collar and mound materials inside mold so that

middle is slightly higher than edges. Attach mold and base plate to pedestal. Place preheated hammer in mold, and apply the appropriate number of blows to top side of specimen.

C-10. Remove the mold from base plate. Place a paper disc on top of specimen and rotate the mold 180° so that top surface is on bottom. Replace mold collar and attach mold and base plate to pedestal. Place hammer in mold and apply same number of blows to bottom as were applied to top.

C-11. Remove paper filters from top and bottom of specimens. Cool specimens and extrude from mold using a hydraulic jack. Place identification marks on each specimen with an alphanumeric code using a grease pencil. Allow specimens to sit at room temperature overnight before further testing.

C-12. Determine the bulk specific gravity for each specimen by weighing in air. Submerge samples in water and allow to saturate prior to getting submerged weight in SSD condition. Remove sample and weigh in air in saturated surface dry condition. This test is conducted in accordance with AASHTO T166.

C-13. Measure the Rice specific gravity on the loose HMA mix samples in accordance with AASHTO T209 (ASTM D2041). This Rice specific gravity is required for voids analysis. If the mix contains absorptive aggregate it is recommended to place the loose mix in an oven (maintained at the mix temperature) for at least 4 hours so that the asphalt cement binder is absorbed by the aggregate prior to testing. Keep the mix in a covered container while in the oven. If the test is run in triplicate on the mix containing near optimum asphalt content, average the three results, calculate the effective specific gravity of the aggregate, and then calculate the Rice specific gravity or TMD for the remaining four mixes with different asphalt contents. If one TMD test is conducted on all each of the five mixes containing different asphalt contents, then

calculate the effective specific gravity of aggregate in each case. Calculate the average effective specific gravity, and then calculate the TMD values for all five mixes.

STEP D. DENSITY AND VOIDS ANALYSIS

Refer to the section on weight volume relationships in this chapter to complete the calculations in the following steps:

D-1. For each specimen, use the bulk specific gravity (G_{mb}) from Step C-12 and the Rice Specific gravity (G_{mm}) from C-13 to calculate the percent voids or VTM.

$$VTM = \left(1 - \frac{G_{mb}}{G_{mm}}\right) 100$$

D-2. Calculate the density of each Marshall specimen as follows:

$$Density\ (g/ml) = Bulk\ Specific\ Gravity\ (G_{mb}) \times (\delta_w)$$

D-3. Calculate the VMA for each Marshall specimen using the bulk specific gravity of the aggregate (G_{sb}) from Step A-2, the bulk specific gravity of the compacted mix (G_{mb}) from Step C-12, and the asphalt content by weight of total mix (P_b):

$$VMA = 100\left(1 - \frac{G_{mb}(1 - P_b)}{G_{sb}}\right)$$

D-4. Calculate the VFA (voids filled with asphalt) for each Marshall specimen using the VTM and VMA as follows:

$$VFA = 100\left(\frac{VMA - VTM)}{VMA}\right)$$

STEP E. MARSHALL STABILITY AND FLOW TEST

E-1. Heat the water bath to 140°F (60°C) and place specimens to be tested in the bath for at least 30 but not more than 40 minutes. Place specimens in the bath in a staggered manner to ensure that all specimens have been heated for the same length of time before testing. Use a waterbath large enough to hold all specimens prepared for the mixture design.

E-2. After heating for the required amount of time, remove a specimen from the bath, pat with towel to remove excess water, and quickly place in the Marshall testing head.

E-3. Bring loading ram into contact with testing head. Zero the pens if using a load-deformation recorder or zero flow gauge, and place gauge on rod of testing head. Apply load at 2 inches/minute (50.8 mm/minute) until maximum load is reached. The maximum load to failure is reported as the stability and the deformation at the point the load just begins to drop is considered to be the flow. Record the stability (maximum load) in lbs. (Newtons) and the flow in 0.01 inches (0.25 mm). Testing should be completed within 1 minute from the time specimen is removed from bath so that excessive cooling of the sample does not occur.

E-4. Repeat Steps E-2 and E-3 until all specimens have been tested being careful to check that:

1. The total elapsed time between removing a specimen from the water bath and the maximum load applied is less than 60 seconds.
2. The total time in the water bath for each set of 3 specimens is the same and is between 30 and 40 minutes.

STEP F. TABULATING AND PLOTTING TEST RESULTS

F-1. Tabulate results from testing, correct stability for specimen height (ASTM D6927), and calculate average of each set of 3 specimens.

F-2. Prepare the following plots on graph paper:

- Asphalt content versus density (or unit weight);
- Asphalt content versus Marshall stability;
- Asphalt content versus flow;
- Asphalt content versus air voids (or VTM);
- Asphalt content versus VMA; and
- Asphalt content versus VFA.

F-3. Review the plots for the following trends:

- Stability versus asphalt content can follow two trends:
  (a) Stability increases with increasing asphalt content, reaches a peak, and then decreases.
  (b) Stability decreases with increasing asphalt content and does not show a peak. This curve is common for some recycled HMA mixtures.
- Flow should increase with increasing percent asphalt cement.
- Density increases with increasing asphalt content, reaches a peak, and then decreases. Peak density usually occurs at a higher asphalt content than peak stability.
- Percent air voids should decrease with increasing asphalt content.
- Percent VMA decreases with increasing asphalt content, reaches a minimum, and then increases.
- Percent VFA increases with increasing asphalt content.

STEP G. OPTIMUM ASPHALT CONTENT DETERMINATION

G-1. The following two methods are commonly used to determine the optimum asphalt content from the plots.

Method 1 - NAPA Procedure contained in TAS 14 (*20*)

a) Determine the asphalt content which corresponds to the specification's median air void content (4 percent typically). This is the optimum asphalt content.
b) Determine the following properties at this optimum asphalt content by referring to the plots:
   - Marshall stability;
   - Flow;
   - VMA; and
   - VFA.
c) Compare each of these values against the specification values and if all are within the specification, then the preceding optimum asphalt content is satisfactory. If any of these properties is outside the specification range, then adjust the optimum asphalt content slightly to bring all of the properties within requirements. If this can not be done then the mix may have to be redesigned.

Method 2 - Asphalt Institute Method in MS-2 (*19*)

1. Determine: (a) Percent asphalt content at maximum stability
   (b) Percent asphalt content at maximum density
   (c) Percent asphalt content at mid point of specified air void range (4 percent typically)
2. Average the three asphalt contents selected above.
3. For the average percentage asphalt content, go to the plotted curves and determine the following properties:
   - Stability;

- Flow;
- Air voids; and
- VMA.

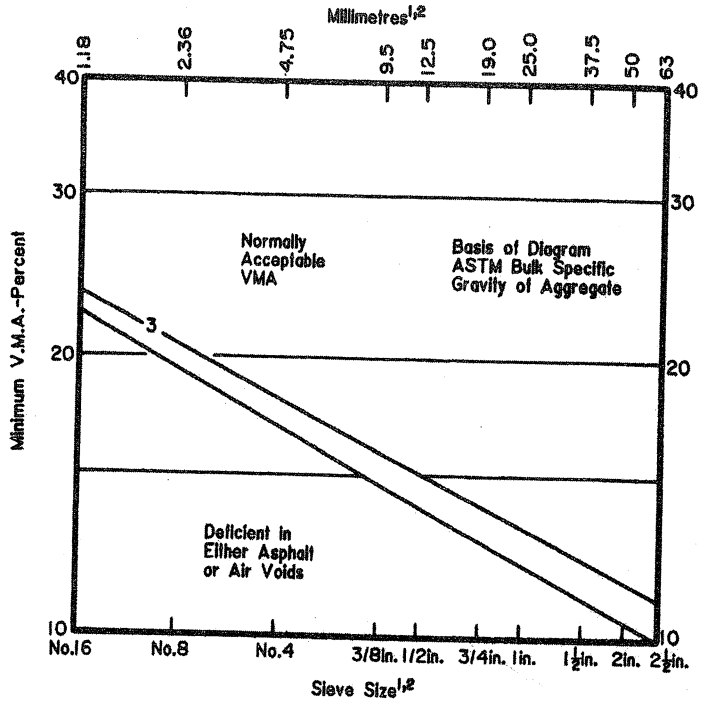
4. Compare values from Step 3 with criteria for acceptability given in Table 4-2 and VMA requirements shown in Figure 4-9.

G-2. Failure to meet the criteria for any property requires that the mix be redesigned or that special action be taken to determine if the mix can be used in construction. Local experience with a mix which fails to meet particular criteria may verify that the mix performs satisfactorily. If that is the case, the criteria may be modified only for those mixes that serve satisfactorily under specified traffic and environments. If the VMA criteria are not met, the aggregate gradation must be modified and the mixture redesigned.

G-3. The laboratory developed mixture design forms the basis for the initial job mix formula (JMF). The initial JMF must typically be adjusted early in the construction project because of slight differences between the grading of samples of aggregates used for conducting the mix design and the grading of those being used in the field. Often, the asphalt content developed in the laboratory is too high for field construction and must be reduced a few tenths of a percent due to the VTM being lower than that designed in the laboratory. The low VTM and low VMA result due to some breakdown of the aggregate during construction resulting in higher amounts of filler and the voids in the mixture becoming closed. The construction experience along with the quality control data during the first few days should be carefully reviewed and the JMF modified if needed to provide a mix that can be satisfactorily mixed, placed, and compacted. Ensure that the produced mix satisfies all volumetric parameters for laboratory compacted mixture and that it meets the requirements for stability and flow. An example of Marshall mix design is given later in this chapter.

Table 4-2. Marshall Mixture Design Criteria for Surfaces and Bases (*19*)

Marshall Method Mix Criteria	Traffic					
	Heavy		Medium		Light	
	Maximum	Minimum	Maximum	Minimum	Maximum	Minimum
Compaction, No. of blows per side	75		50		35	
Stability, lbs.	---	1800 (8000)	---	1200 (5333)	---	750 (3333)
Flow, 0.01 inches	14	8	16	8	18	8
Air Voids, %	5	3	5	3	5	3
Voids in Mineral Aggregate, %	(Graphically shown in Figure 4-9)					



[1]Standard Specification for *Wire Cloth Sieves for Testing Purposes*, ASTM Designation E11 (AASHTO Designation M92).
[2]For processed aggregate, the nominal maximum particle size is the largest sieve size listed in the applicable specification upon which any material is retained.
[3]Mixture in the 1% tolerance band shall be permitted only when experience indicates that the mixture will perform satisfactorily and when all other criteria are met.

Figure 4-9. Relationship between Voids in Mineral Aggregates and Maximum Sieve Sizes with Criteria Superimposed (*19*)

Marshall Criteria

As mentioned earlier, the criteria used by those that have designed mixes using the Marshall method vary considerably. However, in order to include all the criteria in this discussion, the reader must understand that not all these criteria are used by all specifying agencies. Generally most specifiers include requirements for the following properties in a Marshall mixture design: a minimum amount of voids in the mineral aggregates (VMA), a range of acceptable air void contents, a minimum

stability, and a range or minimum flow value. Some states also add a requirement that the percent of voids filled with asphalt (VFA) be within a specified range. Each of these criteria is discussed below.

Voids in the Mineral Aggregate. VMA is the total volume of voids within the mass of the compacted aggregate. This total amount of voids significantly affects the performance of a mixture because if the VMA is too small, the mix may suffer durability problems, and if the VMA is too large, the mix may show stability problems and be uneconomical to produce. The VMA has two components: the volume of the voids that is filled with asphalt, and the volume of voids remaining after compaction that is available for thermal expansion of the asphalt cement during hot weather. The volume of asphalt cement is critical for durability of the mixture. This asphalt cement volume or percent asphalt along with the aggregate gradation determines the thickness of the asphalt film around each aggregate particle. Without adequate film thickness, the asphalt cement can oxidize faster, the films are more easily penetrated by water, and the tensile strength of the mixture is adversely affected. Figure 4-9 shows the recommendation for VMA proposed by the Asphalt Institute (*19*). Notice that as the particle size decreases, the suggested minimum VMA requirement increases. This occurs because the total void space between small aggregate particles is greater than that between large aggregate particles. Therefore, the optimum asphalt content for a dense-graded aggregate mixture with a maximum aggregate size of 2 inches (50 mm) may be as low as 3.0 to 3.5 percent while for a 3/8 inch (9.5 mm) maximum aggregate size the asphalt content may be as high as 7.0 to 7.5 percent. The VMA for any given mix must be sufficiently high to ensure that there is room for the asphalt cement plus the required air voids. Notice that if a specifying agency has both VMA and air void content criteria in the mix design procedure, the agency is also specifying VFA indirectly since VFA is calculated from VMA and air void content. Remember that the VFA is the amount of VMA that is filled by asphalt (VMA – air voids). Many specifying agencies rigidly apply VMA specifications. They should recognize that any VMA specification should

be supported by field performance data. Foster (*21*) has questioned the data from field performance studies conducted by McLeod in the 1950s on which the minimum VMA recommendations are based.

Voids in Total Mix (VTM). The VTM in the compacted dense-graded HMA specimen at optimum asphalt content is suggested by the Asphalt Institute, NAPA, and others to lie between 3 and 5 percent. There are a number of reasons for recommending this void content range. However, this air void content is for laboratory compacted samples and should not be confused with field compacted samples. If the air voids in the laboratory are low the belief is the air voids in place will eventually be low since the compaction effort in the laboratory is established to provide a density similar to the ultimate in place density. If the air voids in the lab compacted samples are high then the belief is that the air voids in-place will remain high after traffic for the same reason.

Low in-place air voids (typically around 3%) result in bleeding of the mixture (asphalt binder being squeezed to the surface under traffic) due to the voids being overfilled and no place to go under additional compaction except out the surface of the mixture. Low air voids also typically result in loss of mixture stability and rutting due to the build up of pore pressure in the mixture under traffic resulting in loss of strength. High in place air voids typically result in more rapid oxidation of the asphalt binder resulting in durability problems. High air voids also result in permeability of the mixture to water resulting in the binder potentially stripping from the aggregate or loss of pavement strength due to the water penetrating underlying moisture sensitive materials potentially found in the base, subbase, and subgrade. High initial in-place air voids will often be reduced from traffic loading but this densification may result in rutting in the wheel path while there may be little or no densification outside the wheel path.

This desired void content must be approached during construction through the application of compactive effort. Generally the in-place air voids should be approximately 7% or less to ensure that the mixture is impermeable to water. There are cases where compaction is difficult and

additional asphalt is added to reduce the in-place void content. This may result in the mixture becoming over compacted under traffic resulting in low voids and thus bleeding and/or rutting. HMA pavement layers generally transfer the load from the surface to underlying layers through intergranular contact. Adequate shear strength is needed in the mixture to ensure that the mix does not fail under the traffic load.

Density. The magnitude of the density achieved during compaction in the laboratory is not as important as how close the density achieved in the laboratory is to the eventual density achieved in the field after several years of traffic. Remember that in the discussion of the development of both the Marshall and Hveem methods, the developers of the laboratory compaction methods applied enough compactive effort to provide laboratory densities which were comparable to field densities reached for the same materials after several years of traffic. Many states now specify the field density requirement as a percent of the theoretical maximum density, hence density can be achieved by increased compaction, by increased asphalt content, by increased filler content, or any method that reduces voids. Therefore it is important how the density is achieved. Is the density obtained by applying compactive effort to force the particles closer together or by filling the voids with asphalt cement or filler? Either of these methods decrease air voids. However, satisfactory compaction effort on a properly designed mixture produces a mixture with high shear strength, while modifying the mixture to reduce in-place voids may provide a mixture with low shear strength with a tendency for high permanent deformation under heavy vertical loads or in areas of acceleration or deceleration such as at intersections.

Since density is usually expressed as a percent of theoretical maximum density, it is easy to see that anything that decreases the in-place air voids will increase the percent density. Notice that increasing percent asphalt cement and the amount of dust in a mix both increase the percent density but do not necessarily have a positive effect on performance. This is one reason that field compaction of Marshall specimens using field produced materials is important. If the mix is

adjusted to decrease in-place air voids the overall quality of the mixture is evaluated to some extent by measuring the voids in the laboratory compacted samples. Increased density due to increased compactive effort will increase shear resistance and overall performance assuming that there is adequate asphalt cement available to prevent durability problems.

In the Marshall mix design procedure, the density varies with asphalt content. Density increases initially as the asphalt content increases because the hot asphalt cement lubricates the particles allowing the compactive effort to force them closer together. The density reaches a peak and then begins to decrease because the additional asphalt cement begins to "fill" the VMA resulting in the asphalt pushing the aggregates further apart resulting in reduced density.

Stability. Marshall stability is defined as the maximum load carried by a compacted specimen tested at 140°F (60°C) at a loading rate of 2 inches/minute (50.8 mm/minute). This stability is generally a measure of the mass viscosity of the aggregate-asphalt cement mixture and is affected significantly by the angle of internal friction of the aggregate and the viscosity of the asphalt cement at 140°F (60°C). Since the viscosity at 140°F (60°C) is primarily affected by the asphalt cement grade, one of the easiest ways to increase the stability of an aggregate mixture is to change to a stiffer grade of asphalt cement. One may also significantly affect the Marshall stability of a mix by selecting a more angular aggregate; i.e., crushed angular aggregates of the same gradation yield higher stability mixes than do aggregates that are rounded or subrounded such as in natural sands and inadequately crushed gravels. It is possible then for the designer to specify certain asphalt cement and aggregate characteristics to produce mixes with either higher or lower stabilities. Anything that increases the viscosity of the asphalt cement will increase the Marshall stability. One may add a small amount of dust into the mixture which will most likely stiffen the asphalt cement binder and thus increase the Marshall stability. Generally, a finer dust will stiffen the binder more than a coarser dust.

Marshall stability and field stability of the in-place mixture are not necessarily related. Practitioners often assume that if a stability problem (rutting) occurs in the field, it can be solved by increasing the minimum acceptable Marshall stability used in design. Experience in the field has demonstrated that this solution does not work. The basic premise for such a solution is: Marshall stability is a surrogate measure for field stability. Since the stability of a mixture in the field is affected by the ambient temperature, types of loading, rate of loading, tire contact pressure, and numerous mixture properties, it is difficult to accept the above premise since the laboratory Marshall stability only takes into account one subset of the conditions mentioned above. Therefore, if field stability is a problem and rutting or similar problem occurs, a comprehensive study should be conducted to determine which of the mixture characteristics or other components in the pavement structure appear to be causing the problem rather than assuming that higher laboratory stability will solve the problem.

The primary use of Marshall stability is in evaluating the change in stability with increasing asphalt content to aid in selection of the optimum asphalt content. The Marshall stability is also useful in measuring the consistency of a plant produced HMA. Because there are large variations in the stability test values, i.e., the precision of the test is very poor, using the Marshall stability as a specification requirement must be done carefully. In fact, ASTM D6927 contains no precision statement even though attempts to gather data for a precision statement have been made periodically for many years.

Flow. The flow is measured at the same time as the Marshall stability. The flow is equal to the vertical deformation of the sample (measured from start of loading to the point at which stability begins to decrease) in hundredths of an inch. High flow values generally indicate a plastic mix that will experience permanent deformation under traffic, whereas low flow values may indicate a mix with higher than normal voids and insufficient asphalt for durability and one that may experience

premature cracking due to mixture brittleness during the life of the pavement.

For 75-blow Marshall designs that are used on interstates or high volume roads or airfields, the flow value is usually specified to be in the range of 8-16. For 50-blow Marshall designs, used on lower traffic volume roads, the flow is sometimes allowed to be as high as 20. When a modified asphalt binder is used in the asphalt mixture the flow test often becomes ineffective in helping to evaluate the quality of the asphalt mixture. In fact, the measure flow is typically very high for mixtures containing modified asphalts and for that reason is typically not used to help evaluate the quality of these types of mixtures.

Percent Voids filled with Asphalt. There are a number of specifying agencies that include percent voids filled with asphalt cement (percent VFA) in addition to the criteria listed above. If a specifying agency includes a VMA requirement and requirements for air voids, percent VFA is a redundant requirement. As discussed earlier, VFA can be determined if the VMA and air voids in the mixture are known.

HVEEM MIXTURE DESIGN METHOD

The basic philosophy of the Hveem method of mix design was summarized by Vallerga and Lovering (*4*) as containing the following elements:

1. It should provide sufficient asphalt cement for aggregate absorption and to produce an optimum film of asphalt cement on the aggregate as determined by the surface area method.
2. It should produce a compacted aggregate-asphalt cement mixture with sufficient stability to resist traffic.
3. It should contain enough asphalt cement for durability from weathering including effects of oxidation and moisture susceptibility.

These elements were combined into a summary statement which forms the basis for selecting the optimum asphalt content in the Hveem method: Use a dense, well graded aggregate with high internal friction and without an excess of fines and add as much asphalt cement as the mixture will tolerate without losing stability. At least 3 percent air void content is desired in the Hveem mix design. In practice most Hveem mixtures are designed to have 4 percent air voids.

Just as with the Marshall Method discussed previously, the procedure is presented in steps to show how the various elements of the design procedure fit together. Steps A through D are very similar to the Marshall method so are not discussed here. The big difference between the two methods of mix design is that a kneading compactor is used to compact samples for the Hveem method whereas a Marshall hammer is used to compact samples for the Marshall. One disadvantage of the kneading compactor is that it is not very portable and thus this makes it difficult to use in the field during production to control the quality of the HMA. For that reason some agencies that used Hveem mix design actually controlled with a Marshall hammer during production.

In the past the Centrifuge Kerosene Equivalent test was used to evaluate the fine aggregate and the surface capacity was determined for the coarse aggregate. This test was primarily done to estimate the optimum asphalt content but it is rarely used today so it is not discussed here. The preparation of specimens and measuring Hveem stability is outlined in Step E; procedures for voids and density analysis are outlined in Step F; and Step G includes procedures for analyzing Hveem mix design test results to determine the optimum asphalt content. An example of Hveem mix design is given later in the chapter.

Details of specimen preparation by California Kneading Compactor and testing by Hveem apparatus are given in ASTM D1561 and ASTM D1560, respectively.

STEP E. PREPARATION OF SPECIMENS AND MEASURING HVEEM STABILITY

E-1. Prepare samples and conduct Hveem tests as specified in ASTM D1560 and D1561.

E-2. Prepare replicate specimens at asphalt contents corresponding to estimated optimum asphalt content and at 0.5% increments up to 1% above estimated optimum and up to 1% below estimated optimum. At least one set of specimens should have enough asphalt to flush after compaction. Flushing occurs when asphalt cement is forced to the outer surface resulting in a shiny surface.

STEP F. DENSITY AND VOIDS ANALYSIS

F-1. Determine the bulk specific gravity of all specimens and Rice specific gravity of all mixes as per Step C of the Marshall mix design method.

F-2. Calculate the percent air voids (or VTM) and density of the compacted specimens as per Step E of Marshall mix design method.

STEP G. ANALYSIS OF TEST RESULTS

G-1. Tabulate results from testing and prepare plots of:

- Hveem stability versus asphalt content
- Density versus asphalt content
- Percent air voids versus asphalt content

G-2. Review trends of each plot for normal behavior:

- Hveem stability should decrease with increasing asphalt content. This plot may vary from being quite flat to quite steep.
- Density will generally increase with increasing asphalt content to a point and then decrease as additional asphalt is added.
- Percent air voids will decrease with increasing asphalt content.

G-3. This step is intended to determine optimum asphalt content. Remember that the philosophy of the Hveem method of mix design is to select the highest asphalt content to provide the highest durability

without exceeding the point of minimum allowable stability. In most cases the optimum asphalt content is determined to be that asphalt content that provides 4% air voids.

G-4. The laboratory developed mixture design forms the basis for the initial job mix formula (JMF). The initial JMF must often be adjusted early in the construction project because of slight differences between the laboratory supplied aggregates and those being used in the field. The construction experience during the first few days should be carefully reviewed and the JMF modified to provide a mixture that meets the volumetric requirements and one that can be adequately mixed, placed and compacted.

SUPERPAVE MIX DESIGN METHOD

Background

The Hveem and Marshall methods of mix design have been used since the 1940s and 1950s. These mix design methods have performed well for many years but with more traffic and heavier loads it was determined in the early 1980s that an improved method of mix design was needed. A method was needed that could be used to rationally design mixtures for various traffic volumes, axle loads and environments. It was hoped that this new mix design procedure would provide tests and models that could use these test results to predict rutting, thermal cracking, and fatigue cracking.

In 1988, the Strategic Highway Research Program (SHRP) was initiated with a primary goal of developing an improved mix design procedure (*7*, *22*, *23*, *24*). The SHRP program was completed in 1993. The effort consisted of hundreds of researchers working on various parts of the research program and it took a great deal of effort to bring all of the results to a cohesive solution in five years. Even though the volumetric portion of the mix design procedure is essentially complete,

additional work is on-going to verify portions of the procedure and to make modifications where needed.

The study primarily focused on asphalt binder evaluation, mix design, and mixture analysis. However, no part of the research effort looked specifically at aggregates. Certainly, the mixture part of the study included aggregates but there was no effort to evaluate aggregate separate from the other mixture components.

Since aggregates are a very important part of the mixture, the SHRP researchers realized that some guidance must be provided. The guidance that was eventually provided was based on a consensus approach by a group of experts on aggregates. Recommendations were made for desired aggregate properties for various traffic levels and for various depths below the surface of the pavement.

At the conclusion of the SHRP Program in 1993, the resulting system contained the following elements: new grading system for asphalt binder (performance graded (PG) grading system discussed in Chapter 2), consensus properties of aggregate, new mix design procedure, and mixture analysis procedures. This system is referred to as the Superior Performing Asphalt Pavement (Superpave) System. After some fine tuning, most State Departments of Transportation (DOTs) adopted the Superpave design procedure very quickly. However there still are no performance tests that have been adopted nationally to fulfill the objective of a mixture analysis system. Many agencies have adopted some type of performance test that is being used as part of their design system but there is not one that is nationally adopted.

Materials

The Superpave mix design procedures provide guidance on selection of aggregates and asphalt binder. The asphalt binder selection was discussed earlier in Chapter 2 and will not be discussed here. A discussion of the aggregate consensus properties as recommended in Superpave is provided below.

The aggregate properties that are specified as a result of the SHRP program are referred to as the consensus properties. These properties include the coarse and fine aggregate angularity, flat and elongated particles, and sand equivalent test. These tests that were adopted by SHRP were primarily chosen from existing tests. Some of these test procedures were already used by many state DOTs to help specify desired aggregate quality. In some cases, Superpave modified the test procedures for aggregates and in other cases the test procedure was not changed but the criteria was modified. Specific recommendations for specification requirements have been made as a part of Superpave for all of these tests.

Specific guidance was provided for angularity of coarse and fine aggregate. The angularity for coarse aggregate is determined by measuring the percentage of coarse aggregate particles with fractured faces. A sample of coarse aggregate is divided into three stacks. One stack contains all the particles with zero fractured faces. The second stack contains all particles with one fractured face, and the third stack contains all particles with two or more fractured faces. The percentage with one or more fractured faces and with two or more fractured faces is then determined. The test method that is used for this test is ASTM D5821, "Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate."

The fine aggregate angularity is measured by determining the amount of voids when the fine aggregate is poured into a cylinder as shown in Figure 4-10. The fine aggregate is poured into the top end of the cylinder and the amount of voids in the loose aggregate is determined. A higher amount of voids in the uncompacted aggregate indicates a more angular aggregate. The test is typically referred to as the fine aggregate angularity test. The test should be conducted in accordance with AASHTO TP 33 Method A "Test Method for Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, & Grading)." The amount of uncompacted voids in the cylinder can be calculated from the following equation:

$$Uncompacted\ Voids = \frac{V - W / G_{sb}}{V} \times 100$$

where,

V = Volume of cylinder, milliliters
W = Weight of loose fine aggregate to fill cylinder, grams
G_{sb} = bulk specific gravity of fine aggregate

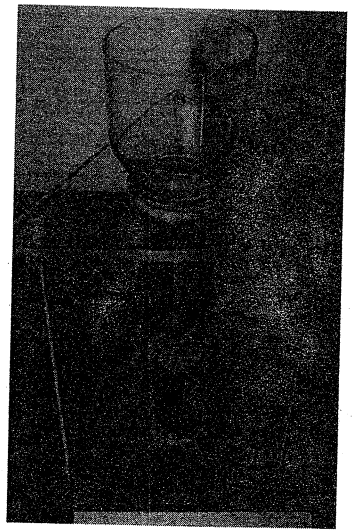


Figure 4-10. NAA Flow Test Set-Up

The test results typically range from the high 30's for natural sands to the low 50's for crushed stone. The criteria was established for the blended fine aggregate and was not intended to be a requirement for each fine aggregate in the blend.

The performance of HMA is related to the angularity of fine and coarse aggregate. When the materials are more rounded, the shear strength is reduced and rutting of the mixture is more likely to occur. The two primary sources of rounded aggregates are gravels and natural sands. Data was developed from a rutting study conducted at NCAT that showed the relationship between aggregate angularity and rutting (25). This data was presented in an empirical model as shown below. This

model was shown to be accurate when the air voids were sufficiently high to prevent plastic flow.

$$Y = (0.080 - 0.000089\,(CF) - 0.00152\,(FAA)) \times \sqrt{ESALs}$$

where,

Y = rut depth in mm
CF = Percentage of coarse aggregate particles with 2 or more fractured faces
FAA = Fine aggregate angularity test for fine aggregates

Another aggregate property that is important for good performance is the test to evaluate flat and elongated particles. When aggregates are flat and elongated, they often tend to lie flat when compacted resulting in a mixture with low voids in the mineral aggregate (VMA). Flat and elongated aggregates also tend to breakdown more during compaction, resulting in in-place mixture properties that may be significantly different from that for the designed mixture. In some cases, mixtures with an excess of flat and elongated particles are difficult to work and compact.

In the past aggregates were classified as flat and/or elongated. An aggregate particle was considered to be elongated if it was significantly longer than wide and it was considered to be flat if it was significantly wider than thick. The Superpave method has combined these terms for flat and elongated particles into one term. The longest dimension of each aggregate piece is now compared to the smallest dimension of that piece to get one number that quantifies the shape of the aggregate. According to Superpave, an aggregate is considered to be flat or elongated if the length divided by thickness exceeds five. The test procedure is provided in ASTM D4791, "Flat or Elongated Particles in Coarse Aggregate."

The final aggregate property that was evaluated by consensus analysis was the clay content. If the amount of clay in a mixture is excessive, a number of problems can result. The most likely problem is the clay adhering to the aggregate and preventing a good bond between the asphalt binder and aggregate. When this occurs stripping is likely to

result in the mixture. To minimize the possibility that this problem or other problems occur, some maximum limit for the amount of clay is specified. The test used to measure the amount of clay is the sand equivalent test. This test can be conducted in accordance with AASHTO T 176 or ASTM D 2419, "Plastic Fines in Graded Aggregates and Soils by Use of the Sand Equivalent Test."

The suggested requirements shown in Tables 4-3 and 4-4 were developed through consensus with a number of experts. The initial recommended requirements are shown in Table 4-3. After some experience and additional research the table was consolidated and the requirements are provided in Table 4-4. These values are believed to be acceptable for most applications of HMA. However, local experience may indicate that the specifications should be different than those values shown in Tables 4-3 and 4-4. Prowell and others documented local experience in aggregate specifications as shown in reference *26*.

Table 4-3. Initial Consensus Properties for Aggregates

Traffic	Coarse Aggregate Angularity, fractured face count *		Fine Aggregate Angularity, uncompacted voids		Flat and Elongated Count, maximum percent with (length / thickness) > 5	Sand Equivalent, minimum
	Depth from Surface		Depth from Surface			
(million ESALs)	< 100 mm	> 100 mm	< 100 mm	> 100 mm		
< 0.3	55/-	-/-	--	--	--	40
< 1	65/-	-/-	40	--	--	40
< 3	75/-	50/-	40	40	10	40
< 10	85/80	60/-	45	40	10	45
< 30	95/90	80/75	45	40	10	45
< 100	100/100	95/90	45	45	10	50
≥ 100	100/100	100/100	45	45	10	50

*First number is minimum requirement for one or more fractured faces and second number is minimum requirement for two or more fractured faces.

There are other aggregate properties besides the consensus properties discussed above that are important to ensure that a quality aggregate is used. Some of these additional properties (source properties) include

L.A. Abrasion, sulfate soundness, and absorption. These test procedures and criteria were not modified by Superpave so they are not discussed in detail under Superpave mix design.

Table 4-4. Revised Consensus Properties for Aggregates

Traffic	Coarse Aggregate Angularity, fractured face count*		Fine Aggregate Angularity, uncompacted voids		Flat and Elongated Count, maximum percent with length / thickness > 5	Sand Equivalent, minimum
	Depth from Surface		Depth from Surface			
(million ESALs)	< 100 mm	> 100 mm	< 100 mm	> 100 mm		
< 0.3	55/-	-/-	--	--	--	40
0.3-3	75/-	50/-	40	40	10	40
3-10	85/80	60/-	45	40	10	45
10-30	95/90	80/75	45	40	10	45
> 30	100/100	100/100	45	45	10	50

*First number is minimum requirement for one or more fractured faces and second number is minimum requirement for two or more fractured faces.

Selection of Aggregate Blend

The applied loads must ultimately be carried by the aggregate. For satisfactory performance an aggregate meeting the quality requirements discussed earlier with the proper grading must be used. The gradation of the aggregate is important to ensure 1) the maximum aggregate size is not too large or too small, 2) VMA requirements are met, and 3) a satisfactory aggregate skeleton is obtained.

According to Superpave, the nominal maximum aggregate size is one sieve size larger than the first sieve to retain more than 10 percent of the material. The maximum aggregate size is defined as one sieve larger than the nominal maximum size (Figure 4-11). If a 19 mm (3/4 inch) maximum aggregate size is specified, then 100 percent of the aggregate must pass the 25 mm (1 inch) sieve size. At least 90-100 percent of the aggregate must be finer than the nominal maximum aggregate size (19 mm or 3/4 inch). Less than 90 percent of the aggregate must pass the

12.5 mm (1/2 inch) sieve. Control points are also set on the 2.36 mm (No. 8) and the 0.075 mm (No. 200) sieve sizes. The aggregate gradation must be within the control limits to meet the Superpave requirements.

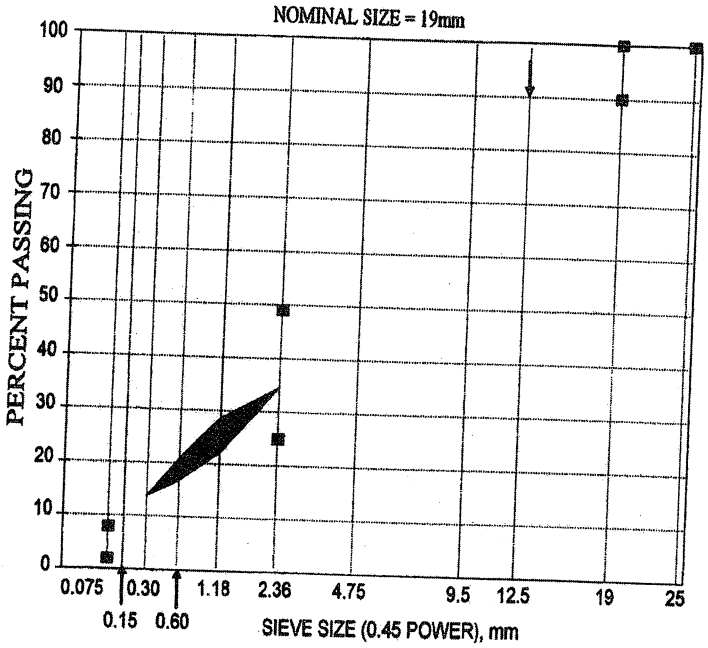


Figure 4-11. Illustration of Gradation Requirements for 19mm Nominal Size

A part of the grading specification that was originally part of the Superpave requirements is the restricted zone (see Figure 4-11). The restricted zone is no longer a recommended requirement but it is still good to use as a guide for those involved in developing mix designs (*27*). The restricted zone is meant to be a guide to help ensure that too much natural sand is not used in the mixture and to help ensure that minimum VMA requirements are met. However, there are aggregates that can pass through the restricted zone that are well crushed and meet the minimum VMA requirements. Aggregate blends that pass through the restricted zone that do not use excessive amounts of rounded aggregates and that meet the minimum VMA requirements should perform satisfactorily.

There is evidence to support the belief that mixtures closer to the low end of the control limits (those going underneath the restricted zone) provide a better stone structure. Initially most Superpave designs were conducted below the restricted zone to provide a mixture that was believed to be more resistant to rutting. However, tests conducted at the WESTrack and at the NCAT test track (*28*) have shown that fine-graded mixtures (mixtures above the maximum density line) are also resistant to rutting as long as high quality aggregates are used. A significant problem that has occurred with coarse-graded mixtures (gradations below the maximum density line) is high mixture permeability (*29*). When these mixtures are not well compacted they tend to be porous resulting in the mixtures absorbing water and staying wet for long periods of time.

A typical specification for a 12.5 mm minimum nominal size aggregate is shown in Table 4-5. The requirements for the restricted zone are shown here even though the restricted zone is no longer part of the requirements. It is shown her for historical purposes since it was part of the initial Superpave requirements.

Typically for mix design, two-three aggregate gradations will be evaluated. Prepare two HMA samples for each of the two-three aggregate gradations (many times a fine-graded and coarse-graded aggregate will be included. The estimated optimum asphalt content is added to these samples and the samples compacted in the Superpave gyratory compactor at the required setting. Several items can be evaluated from compaction of these trial samples. Some of these items include: What is the approximate optimum asphalt content, will the mixture with the aggregate blend being evaluated meet the VMA requirements, and will the gyratory properties such as N_i and N_{max} be met? If some type of performance test is used, how do these mixtures perform when tested for performance? After the initial mixtures have been completed and evaluated, the best aggregate blend can be selected for conducting the mix design.

There are times that the aggregate structure regardless of gradation does not meet requirements for VMA, $N_{initial}$ and/or other properties. This

is a good indication that the source of aggregate is not acceptable. For example, if an aggregate is excessively soft, it will be very difficult to meet VMA requirements. If an aggregate has an excessive amount of rounded material it will likely fail the $N_{initial}$ requirements (definition and requirements provided under Superpave Gyratory Compactor section) and possibly the VMA requirements.

Table 4-5. 12.5 mm (½ inch) Nominal Size

SIEVE SIZE mm	CONTROL POINTS		RESTRICTED ZONE BOUNDARY (NO LONGER USED)	
	LOWER	UPPER	LOWER	UPPER
19	100.0	100.0	--	--
12.5	90.0	100.0	--	--
9.5	--	90.0	--	--
4.75	--	--	--	--
2.36	28.0	58.0	39.1	39.1
1.18	--	--	25.6	31.6
0.60	--	--	19.1	23.1
0.30	--	--	15.5	15.5
0.15	--	--	--	--
0.075	2.0	10.0	--	--

Superpave Gyratory Compactor

A key component of the Superpave mix design procedure is the Superpave gyratory compactor (Figure 4-12). This compaction device has been designed so that the proper number of gyrations can be selected to compact HMA samples to a density similar to that obtained in the field under traffic. Gyratory compaction tends to orient the aggregate particles similar to that observed in the field.

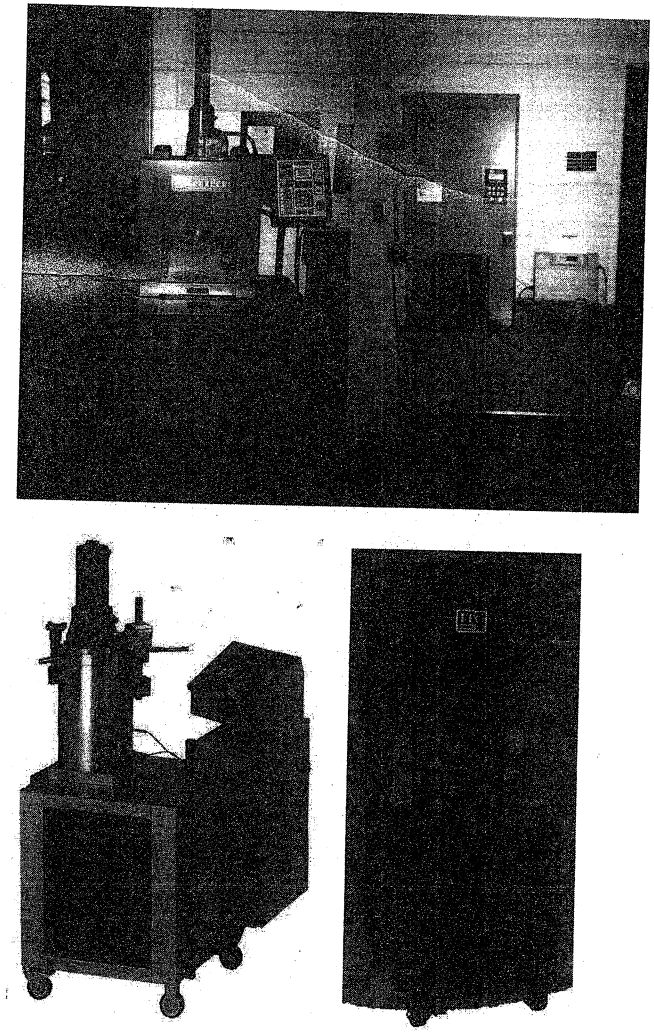


Figure 4-12. Photos of Four Superpave Gyratory Machines

The Superpave gyratory compactor was developed based on evaluation of existing gyratory machines. Although the Superpave gyratory has some similarities to other gyratory compactors, it is a unique piece of equipment. There are three parameters that control the compaction effort on the Superpave gyratory compactor. These settings are vertical pressure, angle of gyration, and number of gyrations. For the

Superpave procedure, the vertical pressure is set at 600 kPa (87 psi) and the angle of gyration is set at 1.25° external angle or 1.16° internal angle. The gyrations are applied at a rate of 30 revolutions per minute. The number of gyrations is selected to provide proper compactive effort to provide a density equal to that expected in the field after several years of traffic.

Early test results indicated that gyratory compactors from different manufacturers generally provided some differences in results. Research indicated that the reason for this difference was the internal angle was different between machines even when the external angles were the same. The differences in results were reduced when the criterion for 1.16° was used for the internal angle *(30)*.

The Superpave gyratory compactor can compact samples 150 mm (6 inches) in diameter or 100 mm (4 inches) in diameter. Most agencies require that samples be compacted to produce 150 mm (6 inches) samples. There are several advantages of using the larger samples. The biggest advantage is that mixtures can be designed using larger aggregates without generating edge effects as a result of the larger aggregate. Generally, the diameter of the sample should be at least 4 times the maximum aggregate size.

The required number of gyrations is based on traffic level. Mixtures that are exposed to higher traffic levels in the field will densify more under traffic and therefore must be compacted to a higher density in the laboratory so that an appropriate asphalt content can be selected. This higher density is obtained in the laboratory by increasing the number of gyrations so that, as mentioned before, the laboratory density is approximately equal to the predicted final in-place density.

The initial specified number of gyrations for specified traffic levels and air temperatures are shown in Table 4-6. These numbers were developed with a limited amount of data so the accuracy of the numbers was not initially known. Research was later funded by NCHRP to consolidate the gyration table. This project was completed and the gyration table was consolidated to that shown in Table 4-7 (*31*). During the consolidation process it was recommended that the effect of air

temperatures is minimal since the PG grade of asphalt is selected to provide the same high temperature stiffness regardless of location. So the stiffness of an asphalt binder in Alabama during the summer would be approximately the same as that in Alaska during the summer. Another study was performed to verify that the gyration levels in Table 4-7 were accurate for the different traffic levels (*32*). Based on this study, it was recommended that the compaction levels in the table be reduced but as the last edition of this book was printed the newly recommended table had not been adopted.

Table 4-6. Initial Number of Gyrations for Superpave Gyratory Compactor

Design ESALs (millions)	AVERAGE DESIGN HIGH AIR TEMPERATURE											
	<39°C			39-40°C			41-42°C			43-44°C		
	N_i	N_d	N_m	N_i	N_d	N_m	N_i	N_d	N_m	N_i	N_d	N_m
<0.3	7	68	104	7	74	114	7	78	121	7	82	127
<1.0	7	76	117	7	83	129	7	88	138	8	93	146
<3	7	86	134	8	95	150	8	100	158	8	105	167
<10	8	96	152	8	106	169	8	113	81	9	119	192
<30	8	109	174	9	121	195	9	128	208	9	135	220
<100	9	126	204	9	139	228	9	146	240	10	153	253
>100	9	143	233	10	158	262	10	161	275	10	172	288

Table 4-7. Consolidated Table for Number of Gyrations for Superpave Gyratory Compactor (*31*)

Design ESALs Millions	N_i	N_d	N_m
<0.3	6	50	75
0.3 to 3.0	7	75	115
3.0 to 10.0	8	100	160
10.0 to 30.0	8	100	160
>30	9	125	205

N_i, N_d and N_m are three terms that need to be identified. N_d is N-design and is the number of gyrations required to produce a density in

the mix that is equivalent to the expected density in the field after several years of traffic. In the mix design process, an asphalt content is selected that will provide 4 percent air voids when the mix is compacted to N_d gyrations.

N_i is N-initial and is a measure of mixture compactibility. Mixtures that compact too quickly are believed to be tender during construction and may be unstable when subjected to traffic. A mix that has 4 percent air voids at N_d should have at least 11 percent air voids at N_i. Mixtures that tend to not meet this requirement are often finer mixtures and often tend to have a large amounts of natural sand.

N_m is the N-maximum and is the number of gyrations required to produce a density in the laboratory that should absolutely never be exceeded in the field. N_d provides an estimate of the ultimate field density. N_m provides a compacted density with some safety factor to ensure that the mixture does not densify too much resulting in low in-place voids thus leading to rutting. The air voids at N_m are required to be at least 2 percent. Mixtures that have less than 2 percent air voids at N_m are believed to be more susceptible to rutting than mixtures exceeding 2 percent air voids.

The number of gyrations for N_d was established based on laboratory and field testing to compare in-place density with laboratory density for various numbers of gyrations. Once N_d was established for each traffic level and air temperature, the number of gyrations for N_i and N_m, was established based on N_d. The equations for N_i and N_m are shown below:

$$N_i = (N_d)^{0.45}$$
$$N_m = (N_d)^{1.10}$$

Compaction of Specimens

Samples of HMA are blended and compacted to produce specimen sizes of 150 mm (6 inches) in diameter and 115 mm tall. These compacted samples are evaluated to determine the volumetric properties as well as the voids at N_i and N_m.

All samples for mix design and volumetric determination are required to be aged at 135°C (275°F) in an oven for two hours after mixing and prior to compaction. This aging is needed to allow absorption of the asphalt cement into the aggregate pores prior to compaction and cooling. This absorption produces a mixture that is more representative of the mix in the field since it is typically stored in a silo at elevated temperature for some amount of time before being loaded into a truck. The sample is spread in a thin layer for the aging process. If the mix is not allowed to age a lower optimum asphalt content will be selected which could result in some loss in durability during the life of the pavement.

When Superpave was first adopted all samples were compacted to N_m and the voids were determined by machine calculations at N_i, N_d and N_m. The sample height was measured and recorded during the compaction process which allowed determination of mixture volume. The sample weight was input into the computer and the sample volume was calculated as it was compacted by knowing the diameter (150 mm or 6 inches) and the sample height. The estimated bulk density was determined from these values. This estimated density is slightly incorrect, however, due to the surface voids on the top, bottom, and sides of each specimen being considered as part of the specimen volume. After each sample was compacted to N_m it was removed from the compactor and the actual bulk density was determined by weighing in air and water. The actual density at N_m was compared to the calculated density at N_m to get a correction factor. The calculated densities at N_i and N_d were corrected based on this correction factor. For example, the correction factor (CF) is calculated on samples compacted to N_m gyrations as:

$$CF = \frac{Actual\ density}{Calculated\ density}$$

The actual density at N_i and N_d can then be determined by:

$$Actual\ Density = CF\ (Calculated\ density)$$

Due to the errors in back calculating the density at N_d, this process is very seldom used any more. The most important density is at N_d which is used to select the optimum asphalt content. Hence, most everyone now compacts to N_d so that the voids can be accurately measured. The density at N_i is often back calculated from the N_d results. If N_m is needed, additional samples are compacted to N_m and the air voids determined. Many agencies no long use N_m since it has been shown that mixes almost never fail and some believe that there is too much emphasis on reducing the optimum asphalt content to prevent rutting. Removing this requirement will seldom change the mix at all and if it does it will change by having a little more asphalt in the mix thus improving durability.

Loose samples of mixture are also used to determine the theoretical maximum density (TMD). By knowing the TMD and bulk density the void level can be determined at the various compaction levels. The TMD samples should be aged in the same way as the samples for compaction.

Selection of Optimum Asphalt Content

After the aggregates and asphalt binder have been determined to meet the quality requirements and after the aggregate proportioning necessary to produce a suitable aggregate structure has been established the optimum asphalt content is determined. The optimum asphalt content has been defined by Superpave as the asphalt content that produces 4 percent air voids at N_d. Of course, this designed mixture must meet the requirements for N_i and N_m and any other requirements that may be specified. After designing the mixture it should be evaluated for water susceptibility using one of the conventional test methods. Superpave currently recommends that AASHTO T-283 be used.

The first step in selecting the optimum asphalt content is to determine an estimate of the optimum asphalt content. Most often an estimate is determined based on past experience but there are times that one may not have experience with certain materials. A good estimate should be available from the design aggregate blend trials. For most

mixes used near the surface of the pavement the optimum asphalt content will be between 5 and 6.0 percent but for many reasons the optimum asphalt content can be considerably different.

After the optimum asphalt content has been estimated it is recommended to prepare three samples each at 0.5 percent below estimated optimum, at estimated optimum, at 0.5 percent above estimated optimum, and at 1.0 percent above estimated optimum. Mix each sample and place in the oven at 135°C (275°F) for the required aging time. Compact each sample up to N_d gyrations based on design traffic and performance temperature.

After each sample has been compacted, remove the sample from the mold and measure bulk density at N_d. The TMD is measured on the same mixture and the voids can be determined. When the TMD is known at one asphalt content, it can be calculated at another asphalt content by determining the effective specific gravity of the aggregate and by making a few calculations as shown in Equation 4-4.

After all of the data has been plotted, pick the asphalt content that will provide 4 percent voids at N_d. Once the optimum asphalt content is determined check the other specification properties such as VMA, VFA, N_i, and N_m to make sure that all specification requirements are met. If all of the volumetric requirements meet then the moisture susceptibility tests are conducted to ensure that the results are acceptable. Another criteria that must be satisfied is VMA. The VMA criteria are shown in Table 4-8.

Table 4-8. VMA Requirements

Nominal Maximum Aggregate Size, mm	Minimum VMA Percent
9.5	15.0
12.5	14.0
19.0	13.0
25.0	12.0
37.5	11.0